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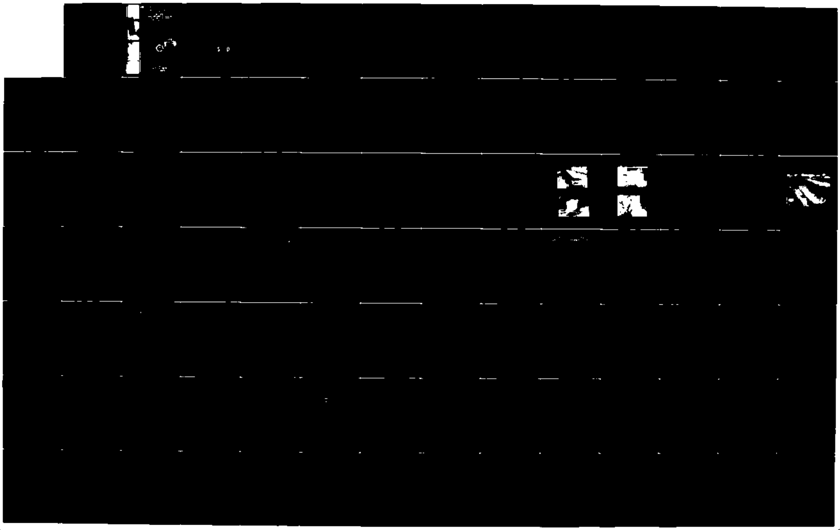

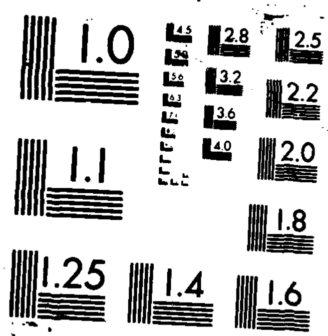

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REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

PROCEEDINGS OF REMR WORKSHOP ON ASSESSMENT OF THE STABILITY OF CONCRETE STRUCTURES ON ROCK

10-12 September 1985

Compiled by

William F. McCleese

Structures Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

January 1987

Final Report

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FIELD	GROUP	SUB-GROUP	Concrete construction; Stability; Research; Rocks

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

Presented are the Proceedings of the Workshop on Assessment of the Stability of Concrete Structures on Rock. The workshop was conducted to define problems with the Corps' current stability criteria and procedures, and to identify research needs that would address these problems. The proceedings provide a summary of the papers presented and the activities, conclusions, and recommendations of five working groups. Each working group was assigned one of the following subject areas.

1. Shear strength selection procedures and the use of these parameters for evaluating the stability of existing concrete structures.

2. Foundation exploration procedures for acquiring test samples and identifying weakness in the foundation for evaluating the stability of existing concrete structures.

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19. ABSTRACT (Continued).

cont'd

3. Computation of forces and methods of analysis for evaluating the stability of existing concrete structures on rock.

4. Instrumentation and monitoring procedures for the purpose of evaluating the stability of existing concrete structures on rock.

5. Procedures for selecting and designing systems to improve stability.

PREFACE

The Proceedings of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program Workshop, "Assessing the Stability of Concrete Structures on Rock," were prepared for the Office, Chief of Engineers (OCE), US Army by the US Army Engineer Waterways Experiment Station (WES). The proceedings provide a record of the papers presented and the reports of the five working groups.

The meeting was organized by WES under the direction of Mr. William F. McCleese, REMR Program Manager, and Mr. Lucian Guthrie, OCE Technical Monitor. Acknowledgements are extended to the following: CPT Wylie Bearup for arranging for the meeting place, video equipment, tape recorders, and paper supplies; each of the speakers who gave a presentation on the first day of the workshop and who furnished a summary for these proceedings; the chairman of each working group who presided over the discussions and presented the findings and conclusions of the working group before the workshop attendees; and the recorders for keeping a record of the working group activities and preparing the report for these proceedings. The proceedings were compiled by Mr. McCleese.

The Workshop was funded by REMR Research Program under Work Unit 32306, "Stability of Existing Concrete Structures on Rock." OCE supervision was provided by Mr. Jesse A. Pfeiffer, Jr., Directorate of Research and Development; and by Messrs. John R. Mikel (Chairman), Tony C. Liu, and Bruce L. McCartney of the REMR Overview Committee. Mr. Lucian Guthrie and Mr. Paul R. Fisher were the OCE Technical Monitors.

Director of WES at the time of the workshop was COL Allen F. Grum, USA. The present Commander and Director of WES is COL Dwayne G. Lee, CE. Technical Director is Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENTS

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
kips (force) per square foot	47.88026	kilopascals
kips (force)	4.448222	kilonewtons
miles	1.609347	kilometres
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

PROCEEDINGS OF REMR WORKSHOP ON ASSESSMENT OF THE STABILITY OF CONCRETE STRUCTURES ON ROCK

INTRODUCTION

The Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Workshop on "Assessing the Stability of Concrete Structures on Rock" was held at the Holiday Inn, Vicksburg, Mississippi, on 10-12 September 1985. The Workshop was sponsored by a research work unit under the REMR program entitled, "Stability of Existing Concrete Structures on Rock." Dr. Carl Pace of the WES Structures Laboratory and Mr. James Warriner of the WES Geotechnical Laboratory are the principal investigators for this work unit.

The stability assessment problem is a multidisciplinary problem which requires the combined efforts of geotechnical and structural personnel for a total solution. A good representation of both groups was present at the workshop. The objectives of the Workshop were:

(1) To promote and establish a good rapport between the geotechnical and structural personnel that would lead to a better understanding of the total problem and a system approach to developing the best possible guidance.

(2) To identify shortfalls in present criteria, procedures, and techniques.

(3) To identify some potential solutions to the identified shortfalls and field input on the areas where research is most needed and most likely to produce significant results.

The first day of the Workshop was devoted to presentations on the experiences, problems, and current practices relating to stability of concrete structures on rock. On the second and third days, attendees were assigned to one of five working groups which met concurrently to summarize existing procedures, identify shortfalls, and recommend potential solutions and directions for future research. The presentations that were given and a record of the activities of each working group are documented in these Proceedings.

ATTENDEES

REMR Workshop of the Stability of Concrete Structures on Rock

Vicksburg, Mississippi 10-12 September 1985

Name	Organization	Group No.	FTS Phone No.	Commercial Phone No.
Agostinelli, Vic	LMVED-TS	5	542-5933	(601) 634-5933
Anderson, Robert	WESGR	4	542-3245	(601) 634-3245
Banks, Don	WESGR		542-2630	(601) 634-2630
Bearup, Wylie	WESSC-A	3	542-3815	(601) 634-3185
Berezniak, Jack	NABEN-F	1	922-4431	
Boggs, Howard	BuRec, Denver		776-4000	(303) 236-4000
Buttrey, Cal	TVA	4	856-3336	(615) 632-3336
Canning, Charles	ORDED-G	1	684-3028	
Chambers, Donald	NPPEN-DB-SA	3		(503) 221-6906
Clough, Wayne	Virginia Tech	3		(703) 961-6637
Deal Hubert	TVA	3	856-3030	(615) 632-3030
DeLoach, Stephen	ETL-TD-EA	4*	385-2816	(202) 355-2816
Dempsey, Lavane	NCDE-S	1	725-7593	
Doak, Sam	NCRED	5		(309) 788-6361
Dowding, C. H.	Northwestern Univ.			(312) 491-4338
Dressler, Don	DAEN-ECE-D	3*	272-8674	(202) 272-8674
Erhart, Joseph	NCBED-DD	3	473-2204	
Foster, Jerry	FERC	3	376-9213	
Gaddie, Thurman	ORDED-T	5	684-2159	
Godwin, Neal Jr.	SWDCO-O	2	729-2429	
Greene, Brian	NCBED-DD	2	473-2241	
Gribar, John	ORPED-DM	1	722-6820	
Groves, Chris	Shannon & Wilson	-		
Gustafson, Lewis	NPDEN-GS	4	423-3867	
Guthrie, Lucian	DAEN-ECE-D	5*	272-8673	(202) 272-8673
Hadala, Paul	WESGV	2*	452-3475	(601) 634-3475
Jackson, Lawson	SWDED-G	2	729-3278	
John, Robert	ORPED-G	2	722-4126	
Johnson, Garrett	NPSEN-DB-ST	1	399-3790	
Kleber, Brian	LMSED-FI	4	273-5638	(314) 263-5638
Kling, Charles	SAMEN-DN	3	537-2635	
Kovari, Kalman	ETU-Zurich	4		
Krysa, Anton	ORPED-DM	5	722-5453	
Lee, M. K.	DAEN-ECE-D	5	272-8676	(202) 272-8676
Lofton, Edd	SWLED-DS	4	740-5161	
Logsdon, Don	NCRED	3		(309) 788-6361
Long, Stuart	ORPED-GG	1	722-4164	
McCleese, Bill	WESSC	5**	542-2512	(601) 634-2512
Munger, Dale	DAEN-ECE-G	3	272-0210	(202) 272-0210

* Working Group Chairman.

** Working Group Recorder.

Name	Organization	Group No.	FTS Phone No.	Commercial Phone No.
Nicholson, Glenn	WESGR	1*	542-3611	(601) 634-3611
Oliver, Lloyd	SAMEN-DG	1	537-3684	
Pace, Carl	WESSC	3**	542-3221	(601) 634-3221
Peters, John	WESGE	3	542-2590	(601) 634-2590
Rahdakrishnan, N.	WESKV	3	542-2187	(601) 634-2187
Riddle, Todd	LMVED-GG	2	542-5886	(601) 634-5886
Schlaht, Larry	MROED	1	864-4485	
Singh, Hari	NCD/WESGR-M	1**	542-3974	(601) 634-3974
Singhal, Avi	WESSC (ASU)	4**		(601) 965-6901
Simmons, Marvin	ORDN-G	1	852-5686	(615) 251-5686
Snipes, Robert Jr.	SAWEN-GS	4	671-4705	
Speaker, John	ORLED-D	1	382-5601	
Staab, Ervill	MRDED-TS	3	864-7222	(402) 221-7222
Strom, Ralph	NPDFN-TE	5	423-3863	
Tracey, Fred	WESKA-E	3	542-3509	(601) 634-3509
Warriner, James	WESGR-M	2**	542-3610	(601) 634-3610
Weaver, Frank	LMVD	1	542-5896	(601) 634-5896
White, John	SPKED-D	3	460-2070	(916) 551-2070
Wright, Richard	NANEN-DF	2		(212) 264-0847
Yost, Bob	ORHED-G	2	924-5234	

AGENDA

REMR Workshop on the Assessment of the Stability of
Concrete Structures on Rock

Conference Room at Holiday Inn

Tuesday Sept. 10, 1985	Presentations	Speaker
8:30 a.m.	Welcome, Announcements, Objectives, and Plans	Bill McCleese
8:40 a.m.	Summary of Experiences, Problems, and Needs of Ohio River Division	Thurman Gaddie
9:10 a.m.	Summary of Experiences, Problems, and Needs of North Central Division	Hari Singh
9:40 a.m.	Survey of Stability Investigations of Concrete Structures on Rock	Larry Schlaht
10:10 a.m.	Coffee Break	
10:30 a.m.	Stability Analysis of Troy Lock and Dam	Carl Pace and Chris Groves
11:10 a.m.	Current Practices of Tennessee Valley Authority	Harold Buttrey
11:40 a.m.	Lunch	
12:00 p.m.	Current Practices of Bureau of Reclamation	Howard Boggs
1:10 p.m.	Current Practices of Federal Energy Regulatory Commission	Jerry Foster
1:40 p.m.	Computer Codes Available to Assist in Stability Analysis	N. Radhakrishnan
2:10 p.m.	Coffee Break	
2:30 p.m.	Experiences in Stability Analysis and Borehole Micrometer	Kalman Kovari

AD-P005 681

USE OF ROCK ANCHORS TO IMPROVE STABILITY OF STRUCTURES WITHIN THE OHIO RIVER DIVISION

Mr. Thurman Gaddie

U.S. Army Corps of Engineers,
Ohio River Division

1. Within the last 20 years, rock anchors have been used to repair fractures and to improve stability of 28 structures within the Ohio River Division. Twenty-two of these applications were for the purpose of assuring stability of both new and existing structures. A tabular summary of these 22 stability applications is presented in Table 1*. The stability criteria used for the design of rock anchors are shown.

2. It will be noted that the stability criteria shown vary substantially. This variation is considered warranted as the criteria were based on the degree to which foundation strengths, geologic conditions and loadings were known or could reasonably be determined. Less conservative factors of safety were used where detailed foundation investigations were conducted (by means of calyx holes, hand-cut foundation block specimens, joint and fault mapping, and extensive laboratory testing) and reasonably conservative failure plane assumptions were employed. Previous maximum loadings on existing structures were carefully considered. Where appropriate, consultants were employed to evaluate design parameters, analyses, and procedures.

3. In summary, considerable engineering judgement went into selecting design criteria that were compatible with the degree of certainty to which other design parameters could reasonable be established. In the author's view, it would be inappropriate for the Corps to establish firm stability criteria for remedial work, without closely relating factors of safety to the methods and procedures for investigating foundation conditions, assigning strengths and numerically analyzing stability.

4. Slides of remedial work were shown and discussed.

---

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

Tuesday Sept. 10, 1985	Presentations	Speaker
3:10 p.m.	New ETL, "Stability Criteria for the Rehabilitation of Navigation Concrete Structures"	M. K. Lee
3:30 p.m.	Measurement for In-Situ Backfill Pressures	Wayne Clough
4:00 p.m.	Adjourn*	

Wednesday Sept. 11, 1985	Working Group Sessions (Total of 5)
8:00 a.m.	Meet in Main Conference Room for assignments and instructions.
8:15-11:00 a.m.	Individual Working Groups meet in separate rooms to derive an outline for their final presentation and report.
11:00-12:00 a.m.	Main Conference Room. Each Chairman will make a 10-minute presentation on the outline and plans of their working group.
12:00-1:00 p.m.	Lunch
1:00-4:30 p.m.	Individual Working Group meetings
4:30 p.m.	Adjourn

Thursday Sept. 12, 1985	Working Group Sessions (Total of 5)
8:00-12:00	Individual Working Group meeting
12:00-1:00 p.m.	Lunch
1:00-3:30 p.m.	Presentations by Chairmen of each Working Group (30 minutes each)
3:30 p.m.	Adjourn

*NOTE: Working Group Chairmen and Recorders will meet for a 15-minute discussion to be led by Avi Singhal.

TABLE 1

APPLICATIONS OF ANCHORS FOR STABILITY

CATEGORY	PROJECT	STRUCTURE	ANCHORS	STRESSED	CRITERIA SLIDING	CRITERIA OVERTURNING	REMARKS
New Navigation Projects	Willow Island	Miter Sills	Vert Williams 1-3/8 Hollow @ 5' OC Ea Way 30^k Assumed	Yes 35^k Lock Off	4.0 Norm 2.67 Maint	100% AB Norm 75% AB Maint	Criteria current at the time
	Racine	Ditto	Ditto	Ditto	Ditto	Ditto	Ditto
	Belle-ville	Ditto	Ditto	Ditto	Ditto	Ditto	Ditto
	Newburgh	Dam Piers	1 on 2 inclined 52 x ½" 7 wire strand 1289^k	Yes 1504^k	1.5 Norm 1.1 Seis 0.1 g	100% AB Norm 75% AB Maint	$\phi_u = 12°$ $c^u = 0$ Below Base
	Cannel-ton	Pier 2	Ditto	Ditto	Ditto	Ditto	$\phi_R = 8°$ $c^R = 0$ Below Base
	Cannel-ton	River Lock-wall	Horz 1-3/8" Stressteel @ 10" OC in 2' Lock Paving 142^k	Yes 150.7^k	4.0 Norm 2.67 Maint	100% AB Norm 75% AB Maint	Riverwall tied to Middlewall Myllenite Below Base
	Bay Springs	Lockwalls	Slightly Inclined 26 x ½" 7 wire Strand 650^k	Yes 750^k	Ditto	Ditto	Anchors used in Lieu of Gravity Lockwall Design
Existing Navigation Projects	Lock 3 Mon Riv	Lockwalls	Vert & Inclined 1¼" d Dywidag Celltite 117^k	Yes 137^k	2.2 Norm 1.8 Maint	90% AB Norm 70% AB Maint	Some Cement Grouted Anchors. Other Variations Also.

TABLE 1

APPLICATIONS OF ANCHORS FOR STABILITY

CATEGORY	PROJECT	STRUCTURE	ANCHORS	STRESSED	CRITERIA SLIDING	CRITERIA OVERTURNING	REMARKS
	Emsworth	Lockwalls	Vert #18s & Dbl #18s Epoxy Coated Re-bars, Cement Grouted 192^k, 384^k	No	2.2 Norm 1.8 Maint	65-75% Norm 67% Maint	$\phi = 45°$ c = 100psi Sliding no problem.
Existing Navigation Projects	Barkley	Miter Sills	1"d Williams Hollow Re-bars Vert 36^k	Yes 36^k + L	4.0 Norm 2.67 Maint	100% AB Norm 75% AB Maint	Criteria Current at the time
	Old Hickory	Ditto					
	Cheatham	Ditto					
	Hilde-brand	Gated Dam	Inclined 30-47° 21 x ½" 7 wire strand 540^k	Yes 607^k	4.0 Norm	100% AB Norm	$\phi = 18°$, c = 0 $750,000 1975
	Galli-polis	Middle Lockwall Pansy Beds	2"d Williams Hollow Re-bars Horz 120^k	Yes 140^k	Designed to resist hydrostatic loads at 0.6 x ult capacity		
	Greenup	Ditto	1¼" d Dywidag Horz 117^k	Yes 137^k	Ditto		
	London	Ditto	Ditto	Ditto	Ditto		
	Win-field	Ditto	Ditto	Ditto	Ditto		
	Marmet	Ditto	Ditto	Ditto	Ditto		

TABLE 1

APPLICATIONS OF ANCHORS FOR STABILITY

CATEGORY	PROJECT	STRUCTURE	ANCHORS	STRESSED	CRITERIA SLIDING	CRITERIA OVERTURNING	REMARKS
Existing Flood Control Dams	Alum Creek	Gated Spillway Monoliths	45° Inclined 52 x ½" 7 wire Strand 1300^k	Yes 1504^k	1.74 Max Pool	100% AB	$\phi = 20.5°$, $c = 0$ $255,000 1975
	Center Hill	Ditto	8-20° Inclined 23 x ½" 7 wire strand 570^k	Yes 665^k	1.5 Max Pool	100% AB	Tan $\phi = .7$ on Cold Joint
Cofferdams	Hannibal	River Arm Stage I Dam	45° Inclined 7 x .6" 7 wire	Yes 200^k	1.1 Max Pool		Cofferdam Monitered PZ's & Defl $\phi = 10°$
Cofferdams	Uniontown	Stage II Dam Coffer	1¼" Dywidag Celltite	Yes			

NOTES:
Norm - Normal conditions
Maint - Maintenance conditions
Seis - Seismic conditions
k - kips
AB - Active Base
ϕ_R - Residual strength
ϕ_u - Ultimate strength
c - Cohesion
L - Losses
OC - On center
g - acceleration
d - diameter
ult - Ultimate
PZ - Piezometer

ADP005682

SUMMARY OF EXPERIENCES, PROBLEMS, AND NEEDS OF NORTH CENTRAL DIVISION

Hari Singh

U.S. Army Corps of Engineers,
North Central Division

1. North Central Division (NCD) has the responsibility of maintaining existing Corps structures located in the Great Lakes areas, along the Upper Mississippi River and its drainage areas, and along part of the St. Lawrence Seaway. Some of these structures are founded on rock, especially those along the Illinois Waterways in the Rock Island District, the St. Lawrence Seaway in the Buffalo District and in the Sault Ste. Marie area in the Detroit District. A great majority of these structures are navigation locks, and the remaining are spillway structures for earth dams.

2. ER 1110-2-100, "Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures," 1973 with changes to 1977, requires in part: "Stability of principal concrete and earth structures should be reviewed, based on current criteria in cases where original design criteria were less conservative." In compliance with this ER, our districts have launched their respective program to evaluate the stability of all their structures on rock foundations. However, because of many other responsibilities, they have not been able to allocate enough resources to complete the evaluations of all of the structures, and only about 25 percent of the structures have been evaluated.

3. The evaluation analyses performed by the districts and reviewed by the North Central Division indicate that a great majority of the structures do not satisfy the current stability requirements. The overturning requirement was the most critical for all of the evaluated structures. The requirements for sliding were satisfied in the majority of cases.

4. Results of the evaluation analysis of three lock structures have been shown in Tables 1 thru 4 and Figures 1 thru 3. These results are typical of all of the evaluations conducted to date in the North Central Division. The

results clearly indicate that the Corps of Engineers' requirements for stability are not satisfied in these three examples. Remedial measures in the form of posttensioned anchors are needed to stabilize the structures. The estimated costs for the remedial measures for the Eisenhower Lock is about $20 million; for the Davis and Sabin Locks the estimated costs is about $10 million. Lockport Lock was reevaluated on the basis of a revised value of coefficient of earth pressure and a less severe criterion for stability as outlined in Table 1 of Draft ETL "Stability Criteria for Rehabilitation of Navigation Concrete Structures" (Appendix B). The revised analysis met the requirements of the Draft ETL; therefore, no remedial action was taken to stabilize the structure when the structure was rehabilitated last year for other structural deficiencies.

5. An in-depth review of the method of analysis, loads considered, and the shear strength parameters used in the evaluations revealed that our methods of computing earth pressures, and the shear strength selection procedure for rock foundations are very conservative.

6. EM-1110-2-2502 (29 May 1961) requires evaluation of earth pressure against structures on a rock foundation on the basis of the at-rest pressure (K_o) condition. This leads to overestimation of the earth pressures in two ways: (a) evaluation of K_o for compacted soils is very complicated and there is not enough information either in Corps manuals or published literature for a reasonable evaluation of this parameter. Designers, therefore, assume a very conservative value for K_o to protect themselves from embarrassment in case failure occurs; (b) the K_o condition does not appear reasonable for evaluation of all structures on rock foundations. Structures which are founded on relatively softer rock will undergo lateral movements under load due to the elasticity of the foundation materials. This lateral movement results in a state of stress in the backfill which is between the active pressure condition (K_a) and K_o. Therefore, it is necessary to develop a method to evaluate K_o for compacted backfill with reasonable accuracy to be used in the evaluation of the structures on hard rock, and a criterion to choose a coefficient of earth pressure between K_a and K_o to evaluate structures on relatively softer rock.

7. The selection of shear strength parameters for sliding stability in Corps projects depends upon the ability and the judgement of the person responsible for exploration and testing. Corps manuals do not provide guidelines on how to select specimens for testing and what failure criteria (peak, ultimate, or residual) should be used in determining design shear strength. In the absence of any guidelines, the shear strength parameters for reevaluation are selected on the basis of the shear strength of intact rock, and the sliding friction of saw-cut specimens. Specimens of grout over saw-cut rock are tested to represent shear strength parameters at the concrete-rock interface. None of the above strength parameters truly represent the actual condition along a potential failure plane. Sliding failure generally occurs along an existing discontinuity; therefore, shear strength parameters of discontinuities should be used in design rather than the shear strength of intact rock and the sliding friction angles of precut rocks. It is, therefore, necessary to provide guidelines on selection procedures for shear strength parameters, selecting test specimens, etc., for discontinuities.

8. Traditional methods of evaluating overturning stability neglects a significant element of stability. It appears reasonable to believe that when a retaining structure tends to rotate about its toe, a surface of rupture (plane or curve) with resistive shear stresses acting along it has to develop in the backfill materials when the backfill materials extend considerably beyond the heel. This resistive force adds stability to the structure and should be considered in an evaluation of overturning stability. North Central Division strongly feels that a research program should be launched to study and evaluate the magnitude of such a resistive force and to incorporate this resistive force into our existing method of overturning stability analysis.

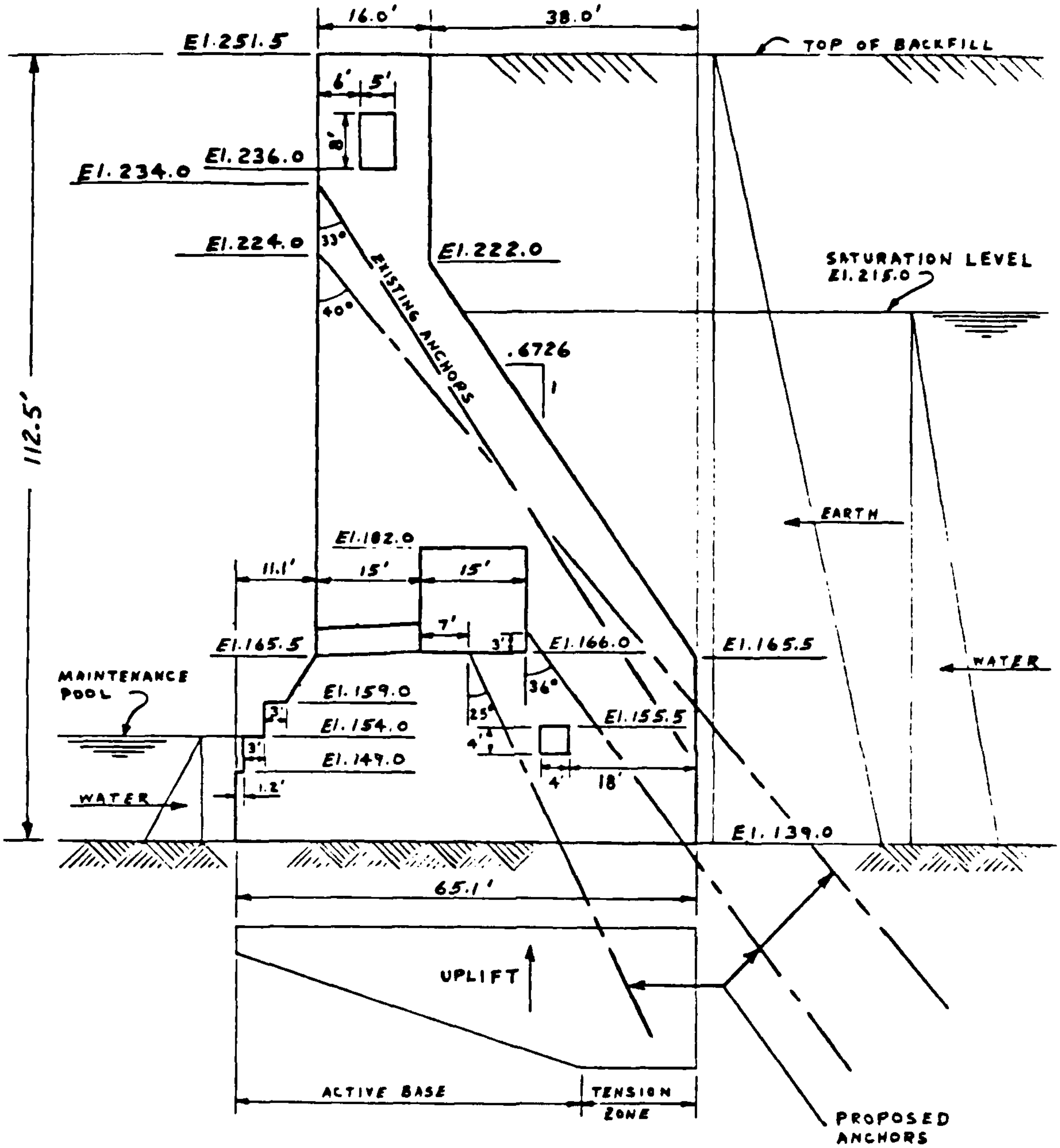
TABLE 1 - Eisenhower Lock: Stability Analysis Summary (Typical Wall between Gates Monolith)

Condition of Loading (1)	Sliding Factors of Safety	Maximum Base Pressure ($K/Feet^2$)	Position of Resultant (2)	% Base in Compression	Number of Anchors (Per 44 Foot Monolith)
II - Existing Normal Low Pool (LP)	1.51*	33.31	3.61 Feet Outside 1/4 Point	58*	
II_A - Existing Normal LP	0.81*	Very Large	6.26 Feet Outside Base	0*	
III - Existing Maintenance	1.22*	41.04	5.75 Feet Outside 1/4 Point	48*	
III_A - Existing Maintenance	0.67*	Very Large	9.82 Feet Outside Base	0*	
II - Anchorage Normal LP	2.00	25.31	2.39 Feet Inside 1/4 Point	86	4 - 600 Kip, Chamber 2 - 605 Kip, Culvert
II_A - Anchorage Normal LP	2.05	39.85	At 1/4 Point	75	4 - 600 Kip, Chamber 16 - 934 Kip, Culvert
III - Anchorage Maintenance	2.00	26.74	4.30 Feet Inside 1/4 Point	95	4 - 600 Kip, Chamber 7 - 609 Kip, Culvert
III_A - Anchorage Maintenance	2.00	42.01	1.02 Feet Inside 1/4 Point	80	4 - 600 Kip, Chamber 16 - 1,160 Kip, Culvert

(1) Cases II and III assume "At Rest" earth pressure (K_o) = 0.36, Cases II_A, and III_A assume "At Rest" earth pressure (K_o) = 0.70.

(2) For a typical monolith the 1/4 point equals 16.28 feet. See Figure 1 for a typical port monolith section.

* - Corps of Engineers' requirements are not met.



EISENHOWER LOCK - NORTH WALL
TYPICAL PORT MONOLITH

CASE III/III$_A$ SHOWN

SCALE: 1" = 20'

FIGURE 1

TABLE 2

LOCKPORT LOCK - STABILITY ANALYSIS

Lock Chamber Land Wall

Case	Bearing Pressure ksf (Ult = 1,991)	Anchor Force Req'd to Meet Overturning Criteria k/ft (80 k/ft provided)	Sliding Factor of Safety Lower Bound Rock Parameters	Sliding Factor of Safety Upper Bound Rock Parameters
I	16.6	61.7	2.8	18.0
II	18.6	60.7	1.8	13.2
III	17.6	40.7	2.4	15.8
IV	18.7	46.9	2.5	16.0

TABLE 3

LOCKPORT LOCK - STABILITY ANALYSIS

Lock Chamber River Wall

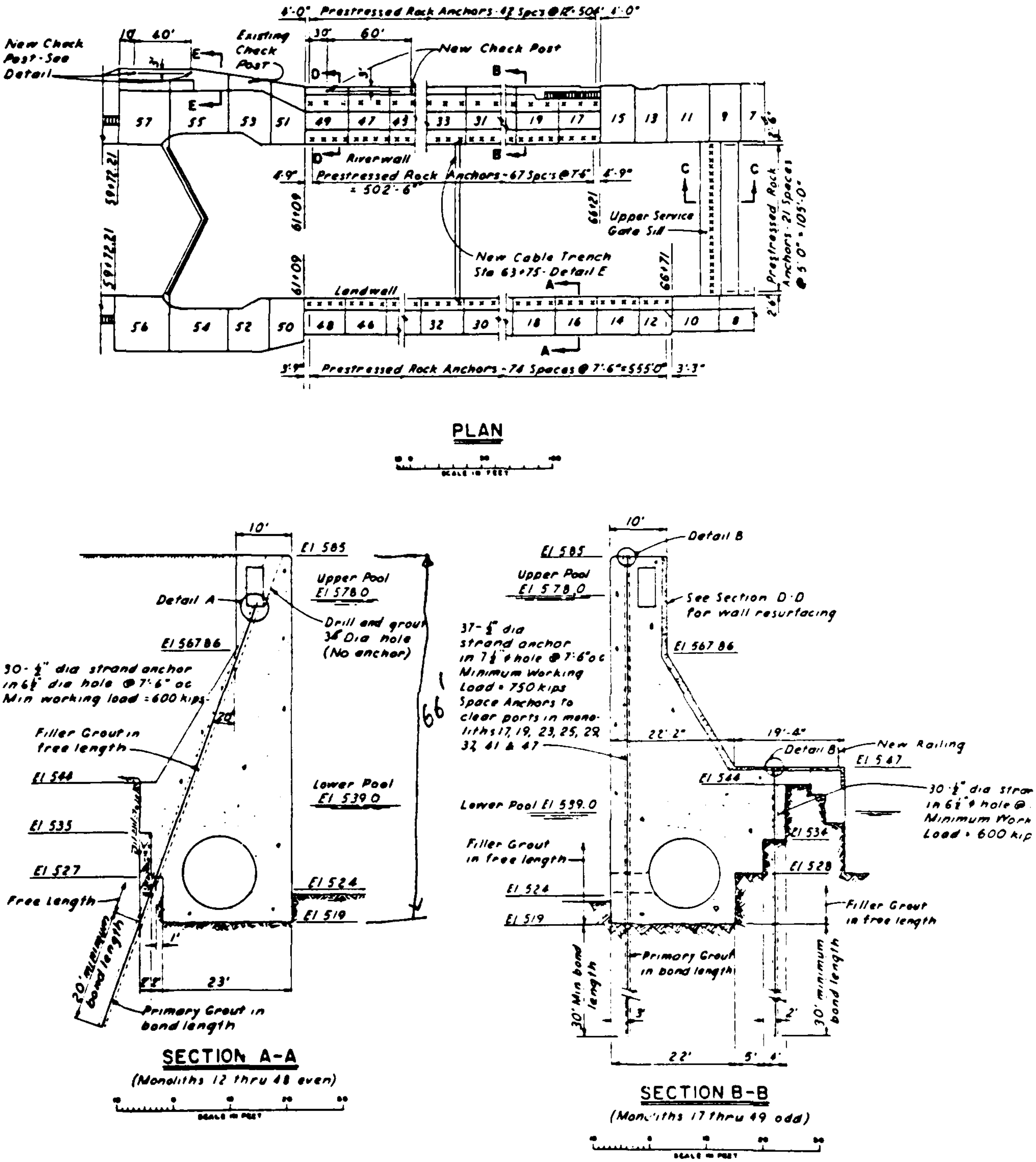
Case	Bearing Pressure ksf (Ult = 1,991)	Base Area (in Compression)	Sliding Factor of Safety Lower Bound Soil Parameters	Sliding Factor of Safety Upper Bound Soil Parameters
I	9.0	100%	1.5	17.4
II	18.0	100%	14.9	153.9

Case I: Normal operating condition. Lower pool in lock chamber at EL.539.00, backfill saturation to EL.549.00.

Case II: Extreme operating condition same as Case I with backfill saturation raised to EL.561.00.

Case III: Extreme maintenance condition, same as Case I, with lock chamber unwatered to lock floor, EL.523.

Case IV: Same as Case I with Earthquake loading.



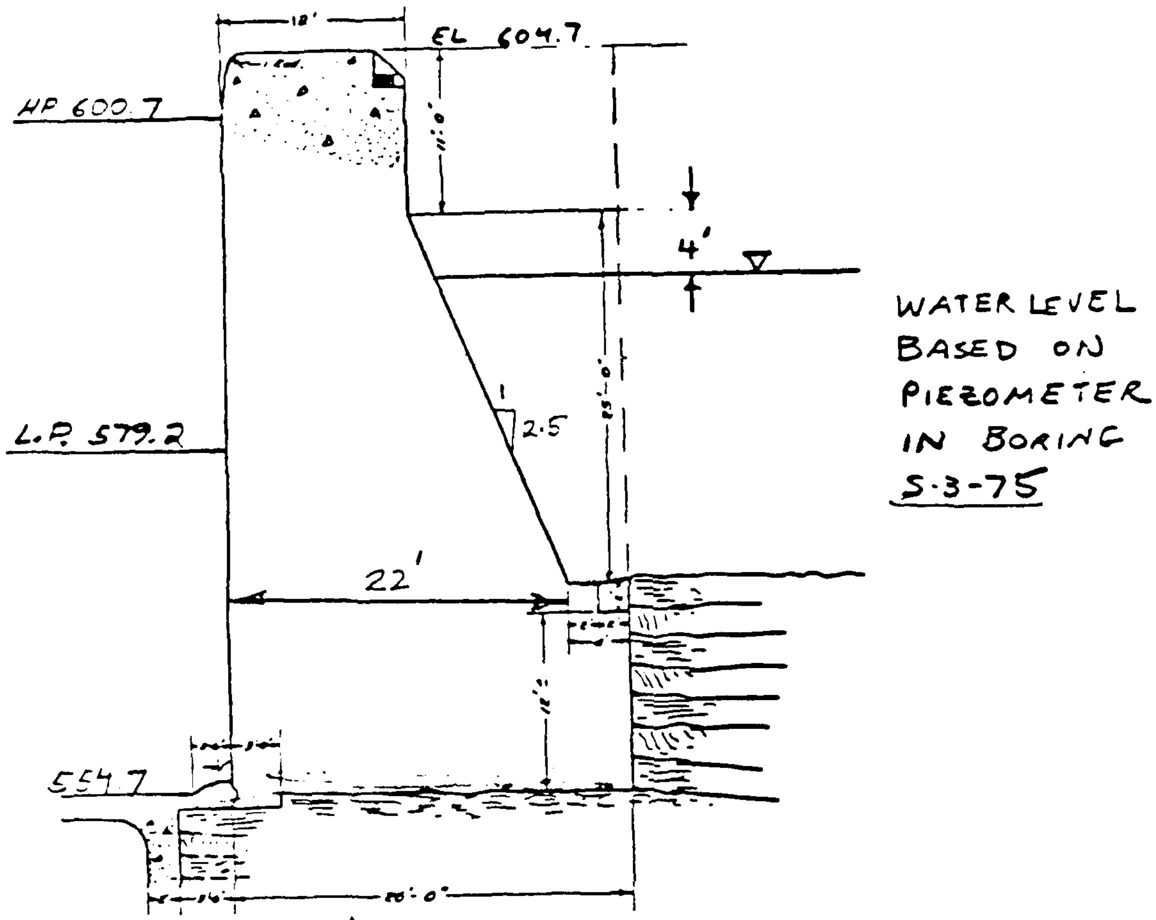
LOCKPORT LOCK
PLAN & TYPICAL SECTIONS

FIGURE 2

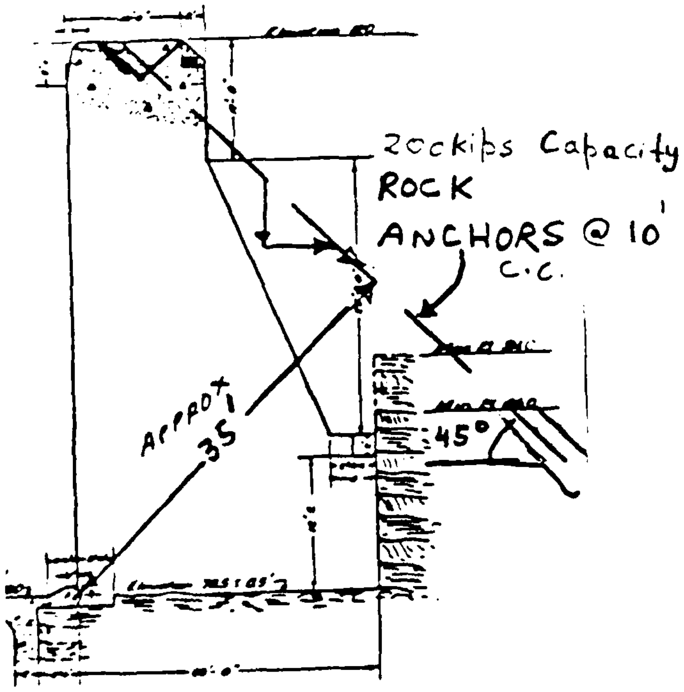
TABLE 4

STABILITY RESULTS, DAVIS & SABIN LOCKS

CONDITION	SECTION	REQUIRE-MENT	RESULTS
Lock dewatered, sliding toward chamber	Narrow wall near upstream end	F.S. ≥ 2	F.S. = 1.25 w/o floor F.S. > 2.0 w/floor
Lock dewatered, overturning toward chamber	Narrow wall near upstream end	Resultant in middle 1/2	3' outside the middle 1/2
Lock at low pool, overturning toward chamber	Narrow wall near upstream end	Resultant in middle 1/3 (kern)	4.8' outside kern
Lock dewatered, overturning toward chamber	Upstream gate monolith used as bulkhead	Resultant in middle 1/2	0.25' within middle 1/2
Lock dewatered sliding toward chamber	Upstream gate monolith used as bulkhead	F.S. ≥ 2	F.S. >> 2
Lock at low pool - uplift	Lock Floor		F.S. may be slightly less than 2
Sabin lock at high pool sliding away from chamber	Narrow wall on north side near downstream end	F.S. ≥ 2	F.S. > 2.0
Sabin lock at high pool overturning away from chamber	Narrow wall on north side near downstream end	Resultant within kern	Resultant within kern



DAVIS & SABIN LOCKS
TYPICAL- SECTION



PROPOSED ANCHORS
TO ASSURE THE RESULTANT
TO BE IN MIDDLE HALF.

FIGURE 3

SURVEY OF STABILITY INVESTIGATIONS OF CONCRETE STRUCTURES [illegible]

AD-P[illegible] 083

Larry Schlaht

U.S. Army Corps of Engineers,
Omaha District

1. Omaha District personnel conducted a survey in early [illegible] pertaining to methods and criteria used on completed and ongoing stability investigations for Corps of Engineers concrete structures on rock foundations. This effort was funded under the Repair, Evaluation, Maintenance and Rehabilitation (REMR) Research Program and a draft report on the survey was completed by the Omaha District in June 1985.

2. Mr. Schlaht's presentation at the stability workshop was a summary of [illegible] survey. He discussed the way the survey was conducted, the response [illegible] survey, and the conclusions reached. The Conclusions and Recommendations [illegible] of the draft survey report are provided below. Most of Mr. [illegible] presentation was drawn from this part of the report. The text in [illegible] duced below) has been modified slightly to omit references to [illegible] of the report.

Conclusions

Criteria References

a. In the course of communicating with many of the District and [illegible] Offices during this survey, it has been sensed that there is not [illegible] some confusion, but also dissatisfaction involving methods and applicable criteria for the stability of concrete structures on rock foundations.

b. Some of the confusion may come from the fact that there are numerous references within the Corps of Engineers on stability criteria, depending on the type of structure. Other sources of confusion may be the result of changes made in the criteria. For instance, gravity dam design stability criteria were formerly given in EM 1110-2-2200 in 1958, then modified by ETL 1110-2-63 in 1969, modified further by

ETL 1110-2-184 in 1974, and finally modified to the current procedures by ETL 1110-2-256 in 1981.

c. Some incontinuity has also evolved by the changes. The last change of criteria (ETL 1110-2-256) in 1981 changed the method and required factor of safety for determining the sliding stability for all concrete structures on rock foundations and left the designer to refer to a particular previous reference for other criteria such as overturning.

Overturning Criteria

d. For those structures which were found to be inadequate for overturning criteria, the evidence of this survey which includes opinions from various other Corps offices indicates that the majority of the inadequacies probably are the result of more conservative uplift assumptions used for the reevaluation as compared to the original analyses. Most uplift assumptions for reevaluations assumed a straight-line distribution at the base of the structure varying from full headwater to full tailwater pressure and the pressure was assumed to be acting over the entire area of the structure under consideration. Most of the older original design analyses generally assumed values which were about fifty to sixty-seven percent less than the reevaluation assumptions.

e. Some of the structures which were found to be inadequate for overturning, were simply the result of using a higher seismic coefficient than that used in the design. For one of these structures, a wave force with a magnitude ten times greater than that used in the original design was also applied.

Sliding Criteria

f. For the structures which were found to be inadequate for sliding criteria, it is assumed that the principal reason for the deficiency is that the shear strengths used in the reevaluations were substantially below those used in the original design assumptions for the cases where either shear friction formula or limit equilibrium analyses were computed. It is noteworthy that for two projects, the lock and spillway structures at the Troy Lake projects and the lock chamber walls for the Lockport Lock project, the required factors of safety were lowered when using lower bound strengths and raised, in the case of the Lockport Lock, when using upper bound strengths. Even these changes in required factor of safety did not keep these projects from being deficient in sliding stability[1].

Recommendations

g. The following recommendations are made with the objective of improving the overall system of stability investigations procedures and criteria:

(1) Consider combining all or most structural stability procedures and criteria into one reference document to eliminate confusion.

(2) Consider additional research using instrumentation data to determine the possibility of using less severe uplift assumptions for reevaluation analyses.

(3) Consider changing the criteria for the required factor of safety to allow more flexibility for variations in shear strengths and loading conditions. As an example, it may be desirable under certain circumstances to require factors of safety for different shear strength assumptions such as upper bound shear strengths and lower bound shear strengths (residual) similar to that done at the Lockport project. Additionally, consideration should be given to requiring a reduced factor of safety for maximum reservoir conditions in addition to the factor of safety for normal

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[1] For the Troy Project, this statement is based on the stability analysis conducted by WES. Shannon&Wilson, Inc. later performed a stability analysis of the Troy Project and their conclusions are included in the presentation by Mr. Chris Groves (page 36).

or usual conditions and for earthquake or extreme loading conditions. Reduced factors of safety for lower shear strengths have been used at the Troy Lake project and the Lockport project. Also, reduced factors of safety have been used by the St. Paul District and Pittsburgh District.

(4) Consider adding more discussion on the selection of shear strengths in any document where criteria are revised. Included should be a thorough discussion on the determination of shear strengths accounting for the effect of deformation or strain incompatibility; type of testing; geological input in the selection of shear strengths; and effect of the confidence in the drilling, sampling, and testing programs on the selection of design shear strengths. It also would be appropriate to stress the importance of the selection of the design shear strength and to adopt a selection criteria based on the confidence level of the testing program.

(5) Consider the presentation and listing of computer programs which are acceptable for analysis of structures in revised stability criteria documents.

(6) Consider additional research of other owners of structures, such as the Bureau of Reclamation and others, to determine whether they have had similar problems of inadequacy under reevaluation processes.

STABILITY ANALYSIS OF TROY LOCK AND DAM

Carl Pace

Structures Laboratory
US Army Engineer Waterways Experiment Station

1. Troy Lock, Dam, and Powerhouse are located on the Hudson River in upstate New York (Figure 1) 156 miles from New York Harbor. Troy Lock and Dam allows entrance to the New York Barge Canal which connects to the Great Lakes. This makes Troy Lock and Dam an important link for shipping and pleasure craft in the northeast.

2. Surface concrete of Troy Lock and Dam is in a deteriorated condition (Figures 2, 3, 4, and 5). Because of this deterioration, the New York District decided in 1978 to have the Structures Laboratory of the Waterways Experiment Station (WES) evaluate the condition of Troy Lock and Dam and determine what rehabilitation should be performed.

3. The first phase of the study consisted of a condition survey where: (a) cracks were mapped, (b) soniscope and impact hammer measurements were taken, (c) construction drawings were reviewed, and (d) operation and maintenance problems were discussed. From the first phase of the study, it was concluded that internal cracking in the lock and dam was structurally insignificant and that the interior concrete was probably in sound condition.

4. The second phase of the study involved a coring program, testing and evaluation of the cores, stability analysis, and stress analysis. The coring program was fairly extensive as shown in Figure 6. Both horizontal and vertical cores were obtained from the lock and dam. Vertical cores were taken in the backfill behind the landwall to obtain samples from which an estimate of the horizontal backfill pressure coefficients could be obtained. The vertical core which was taken through the lock and dam and approximately 25 ft into the foundation showed that the foundation material was very uniform with steeply dipping bedding planes. No weak planes in the foundation (which could cause stability problems) were indicated.

5. The cores from the concrete were used to obtain profiles of depths of deteriorated concrete, and the concrete and foundation core were tested to obtain strength data. Direct shear tests were performed on the concrete and shale core which were located at or near the structure-foundation interface.

6. Soundings were taken downstream of the dam to determine if there were any scour areas which would affect the stability of the dam monoliths. The dam was constructed in 1915. Figure 7 shows that downstream strut resistance does not exist. Rock anchors extend from the dam monoliths into the foundation to help stabilize the dam and were assumed in the stability evaluation to add resistance forces.

7. The compressive strength of the foundation material was found to be weaker than the concrete at the structure-foundation interface. The unconfined compressive strength of the foundation material was only 900 psi and the tensile strength was 43 psi. The lock and dam is not high and the low bearing pressure of the foundation was not a significant problem. The compressive strength of the foundation core increases substantially with confining pressure.

8. Conservative values of $\phi = 30° 24'$ and $c = 0.04$ ksf were used in the stability analysis. These values were obtained from direct shear tests on cut surfaces of shale and concrete close to the structure-foundation interface. If the interface of the structure and foundation (slaty shale) is irregular and some of the slaty shale material has to be sheared for the structure to slide, the ϕ and c values are higher than those used. The shear strength parameters for the slaty shale is $\phi = 42$ to 50° and $c = 10$ to 230 psi.

9. I felt more comfortable in reducing the safety factor for sliding than in increasing the ϕ and c parameters. I used safety factors that were one half of those stipulated in the Corps' engineering manuals. These were 4 for all case loadings on existing structures without earthquake and 2-2/3 for normal operation with earthquake.

10. There were several reasons why I used conservative values for ϕ and c and reduced safety factors. In general, it was because of uncertainties in the

evaluation. Some of these uncertainties are discussed in the following paragraphs.

11. The irregularities at the interface of the structure and foundation are not known. Even though the coring program was extensive, the core holes were too far apart to determine with any degree of certainty the asperities of the structure-foundation interface. It would have been too expensive to expand the coring program to define the asperities with any degree of certainty.

12. Secondly, lock and dams generally vibrate to some degree due to the passage of water. This is true of Troy Dam even during normal operating conditions. In fact, when I was in the gallery of the dam, it seemed to be vibrating so badly that I thought for a minute it was going downstream. I know that vibrations are not usually as bad as they seem and I think this is true at Troy Dam. I did not know and do not know what the vibrations mean in relation to stability. The vibrations which I experienced were during normal operation; in times of ice passing over the dam or some other abnormal condition, the vibrations could be more severe.

13. I do know something about how vibrations can affect the sliding stability of a concrete block. I had a vibrator mounted on the top of a 28,000-lb concrete block sitting on a concrete floor. Without vibrations it took 10,000 lb of static load to slide the block. With the vibrations at the natural frequency of the block a static load of only 250 lbs moved the block. The vibrations reduced the failure load by a factor of 40. Vibrations at a dam are not likely to be at the dam's natural frequency; however, they do not have to reduce the sliding resistance by a large factor to cause problems. I did not know then and do not know now what effect the vibrations may have on the sliding safety factor of Troy Lock and Dam.

14. Other uncertainties included the effects of eccentric loading on shear resistance, uplift forces on the structure, and backfill pressures on the landside lockwall. Since funds were not available to investigate all these uncertainties, I used conservative shear strengths with a logical reduction in sliding factor of safety.

15. The stability analysis indicated that some landwall monoliths of the lock did not meet the criteria for overturning and a lot of the lock and dam monoliths did not meet the sliding stability criteria. Reaction blocks and posttensioning were designed to add additional stability to those monoliths.

16. Reaction blocks were chosen to add additional resistance to sliding because they do not stress or change the structure in any way. Posttensioning a structure could cause stress concentrations in the structure, especially around block-outs, culverts, or other areas where changes in geometry exist.

17. In my report to the New York District, I recommended that only 15 to 20 percent of the required force be applied in the posttensioning strands which were designed to strengthen the monoliths against overturning. This would prevent the addition of large loads to the strands and the structure at times when they are not needed. A slight movement of the structure would add the necessary loads to the posttensioning strands to prevent overturning of the structure.

18. Even though I designed systems to add stability to the monoliths at Troy Lock and Dam, I believed the structure to be stable if scour holes were filled. This was only a belief; therefore, conventional analysis had to be followed. I would like to have had sufficient funds to research uncertainties, do parameter studies, and try to better define the stability of Troy Lock and Dam.

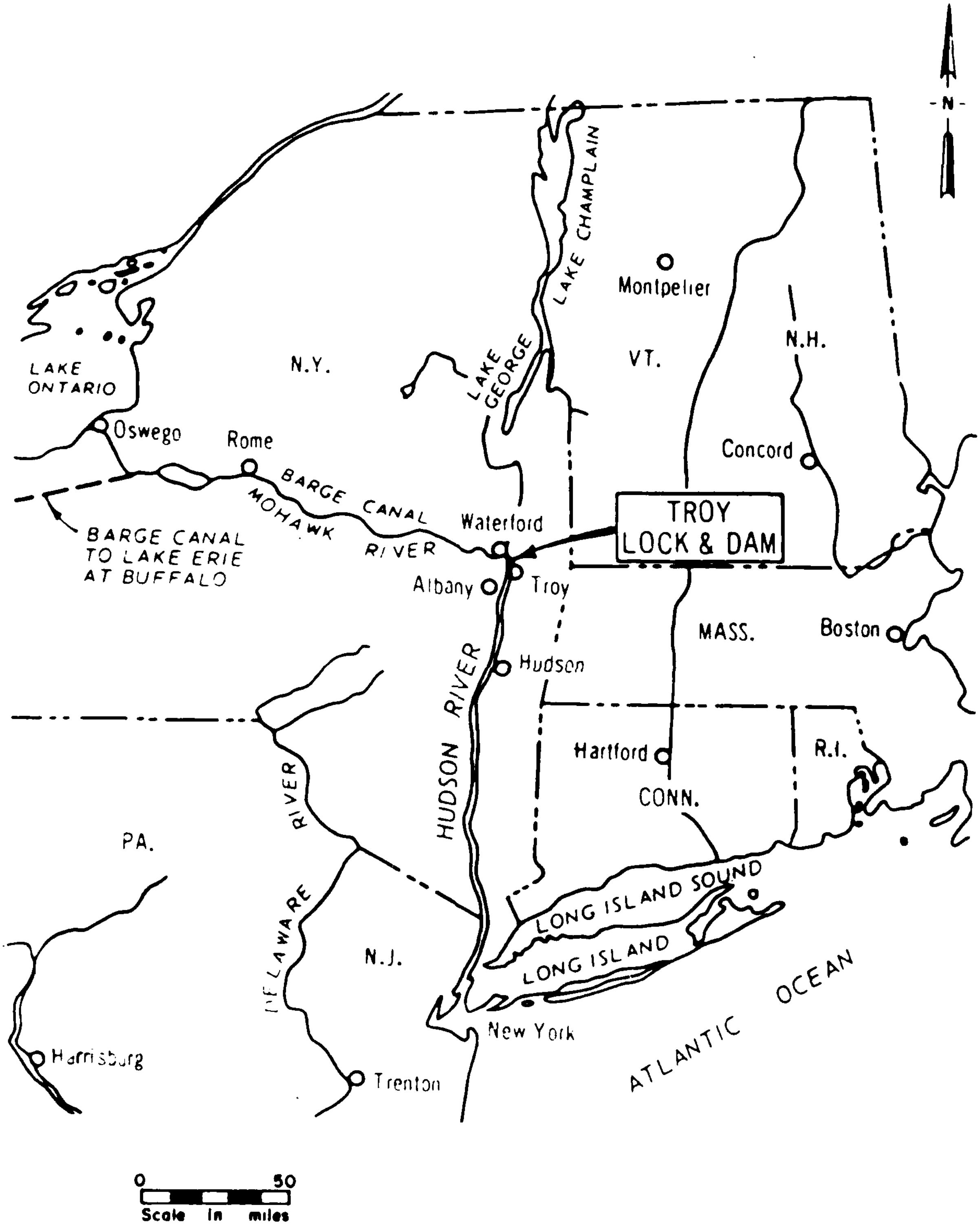

Figure 1. Geographical location of Troy Lock and Dam

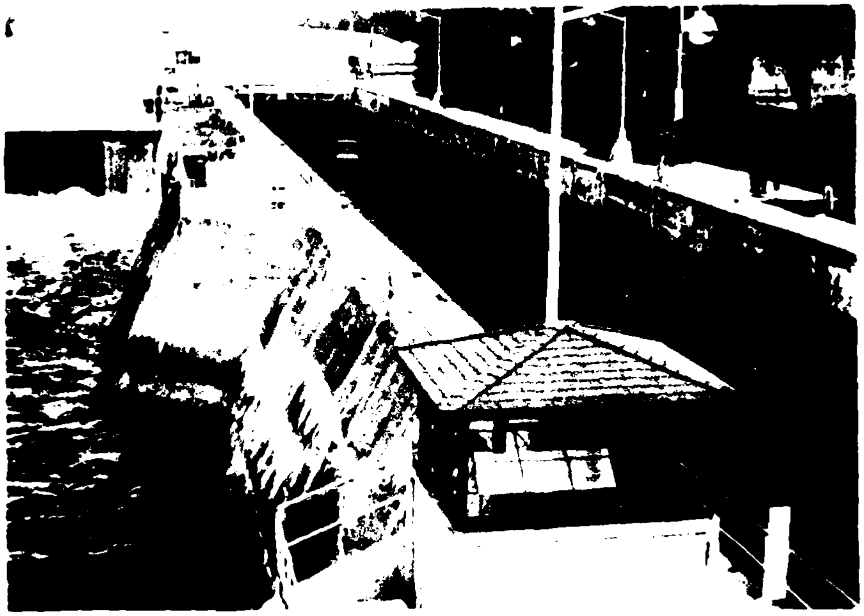

Figure 2: Overview of upstream portion, Troy Lock

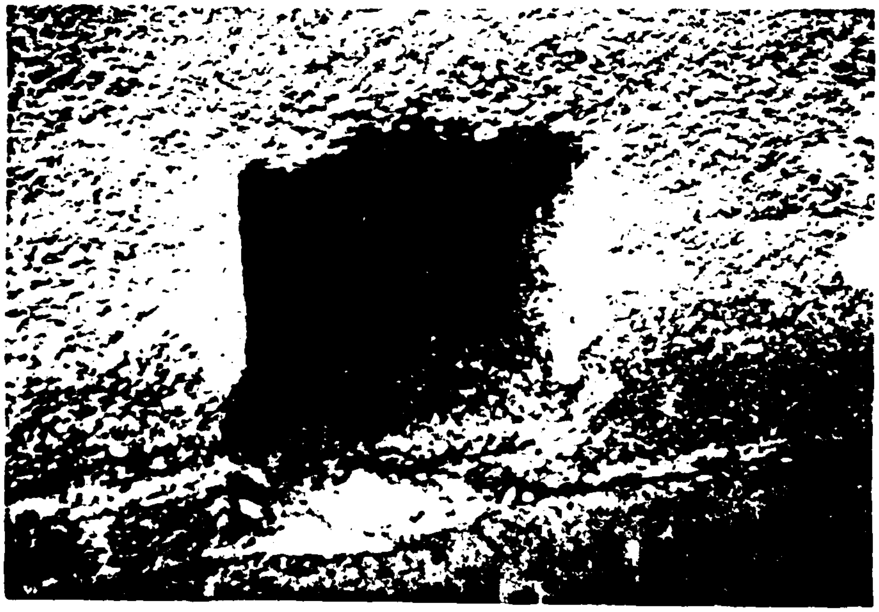

Figure 3: Typical view of filling and emptying culverts

Figure 4: River wall-location where dam joins river wall

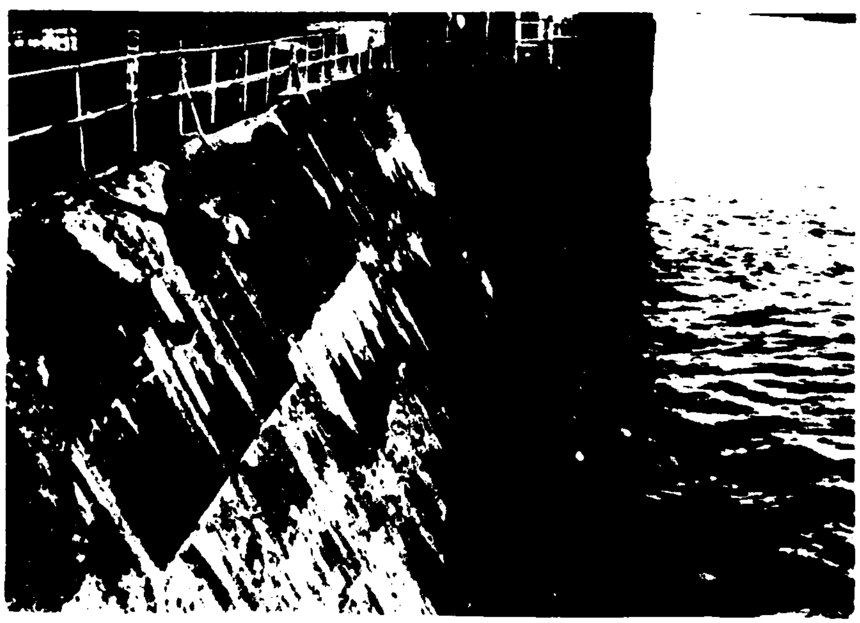

Figure 5: Riverside of river wall

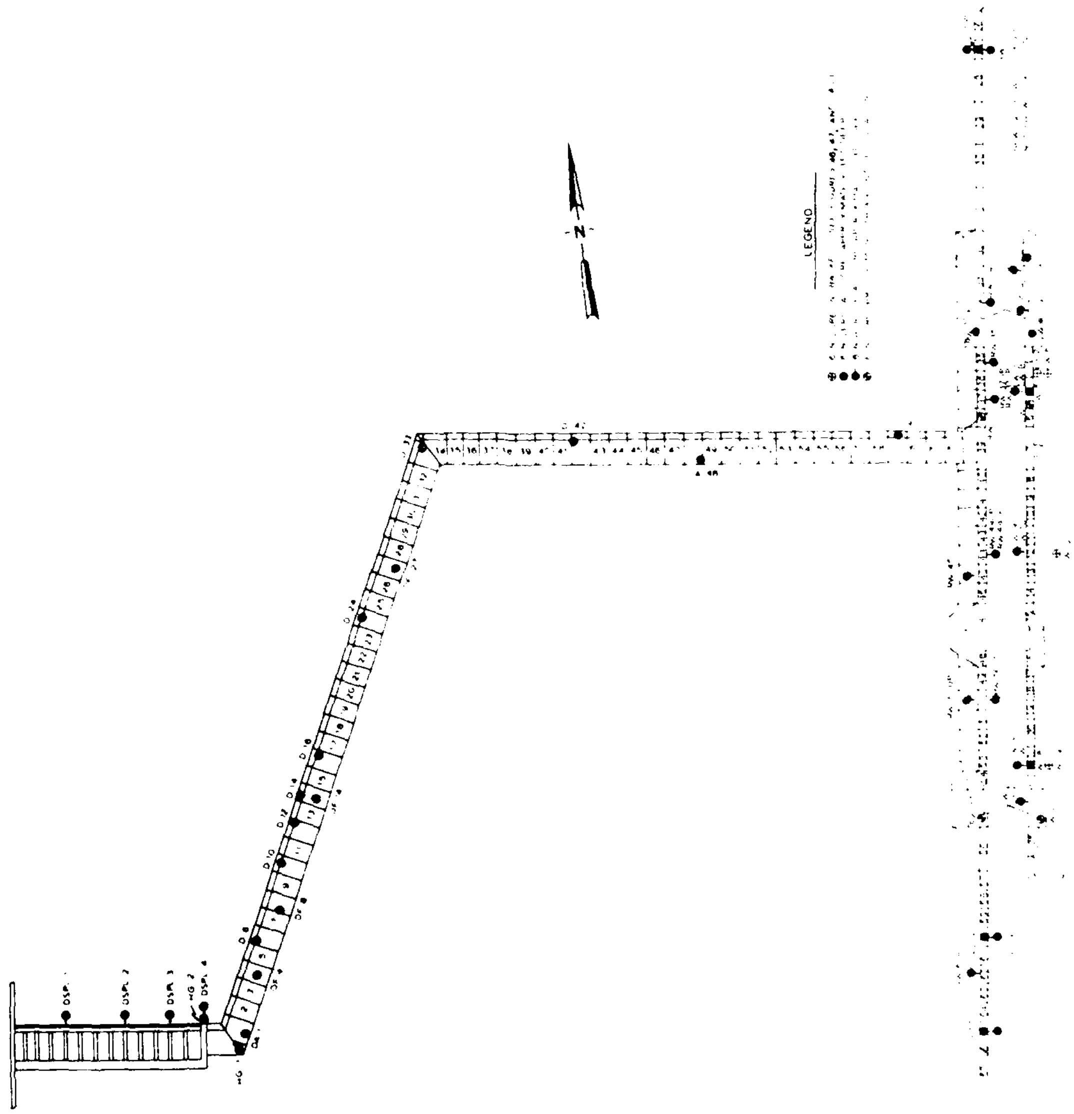

Figure 6: Overall view presenting core locations in the backfill, lock, dam, and headgate section

REVIEW OF METHODS OF ANALYZING THE
STABILITY OF CONCRETE STRUCTURES ON ROCK FOUNDATIONS

ADP005685

Chris Groves

Shannon & Wilson, Inc.

Summary

Mr. Groves discussed the findings and conclusions of Shannon & Wilson, Inc., in their stability evaluation of Troy Lock and Dam. He used slides of some of the photographs and plates contained in their report "Instrumentation Performance and Stability Evaluation, Troy Lock and Dam," August 21, 1985.

The following paragraphs are from the report transmittal letter to the New York District:

"This report presents the results of the study which evaluates the physical parameters that significantly affect the structure's stability. It also includes the instrumentation and testing conducted to evaluate these parameters, the stability analyses, and recommendations which were developed for corrective measures.

The instrumentation program indicates that there are no significant structural movements of the lock and dam, and that the uplift pressures along the structures are linear as assumed in Corps manuals and in their stability analyses. We have determined that the critical failure mode for sliding is through a hypothetical stress relief joint which is assumed to run completely across the structure a short distance into the foundation. A foundation friction angle of 45° was used in the stability analysis. This is higher than the value used in the W.E.S. analysis, but still conservative in our opinion. In our opinion, the analysis confirms that the lock and dam have adequate factors of safety and are stable for all loading conditions in sliding and overturning with one exception. The exception is the section containing lock Monoliths L-4 through L-7 which has 38 to 49 percent of its base in compression during static loading conditions, whereas 100 percent is normally required. Safety factors of 2.7 to 3.0 were calculated for overturning. In addition, these monoliths have a low (1.6) overturning safety factor during earthquake loading.

There are several options for dealing with these four monoliths including 1) accept them as they are, 2) conduct additional analyses, and 3) improve them structurally such as with shallow tiebacks. We recommend that an assessment be made of these options along with cost studies. In view of the excellent performance of the structure, the risk associated with Option 1) may be

Figure 7: Original construction, Troy Dam

acceptable. If the cost of Option 3) is favorable, the installation of several tiebacks may put the matter to rest.

The report also includes recommendations for repair of the structures, suggestions for ongoing monitoring of existing instrumentation, and precautions which should be followed during the repair program."

The Conclusions and Recommendations sections of the Shannon and Wilson, Inc. report on Troy Lock and Dam is reproduced below for ready reference.

CONCLUSIONS

Considerations for Acceptable Stability Criteria

ETL 1110-2-256, Sliding Stability for Concrete Structures, dated 24 June 1981 emphasizes the importance of selecting the appropriate laboratory test for the probable mode of failure. It specifies minimum required sliding factors of safety of 2.0 and 1.3 for normal static loading and seismic loading conditions, respectively. The same values were used by WES in its evaluation of Troy Lock and Dam, and they are the values which we have adopted. Note that ETL 1110-2-256 superseded ETL 1110-2-184, Gravity Dam Design Stability dated 25 February 1974 and, it is assumed the minimum sliding factor of safety contained in ETL 1110-2-22 Design of Navigation Lock Gravity Walls dated 19 April 1967 no longer applies. The latter two ETL's had minimum required sliding factors of safety for normal static loading and seismic loading conditions of 4.0 and 2.67, respectively.

Regarding the minimum acceptable percent of base in compression, ETL 1110-2-22 requires 75 percent for both the normal and dewatered operating conditions, assuming at-rest earth pressures. When considering earthquake loading along with normal operating conditions, the base pressure resultant must remain inside the base and the allowable foundation pressures must not be exceeded. We have no basis for suggesting revisions to these criteria.

Sliding

Based on our analyses, the minimum factors of safety for sliding of the lock walls and the dam monoliths based on the available data are as follows:

Case	Factors of Safety Minimum Computed
1) Normal Operation	
Landside Monolith (L-4 through L-7)	2.4
Riverside Monolith (R-36)	2.7
Dam Monolith	2.2
2) Dewatered Case	
Landside Monolith (L-4 through L-7)	2.2
Upstream Riverside Monolith (R-36)	1.9
3) Normal Operation & Pseudo-Static Earthquake	
Landside Monolith (L-20)	2.0
Riverside Monolith (R-36 and R-50)	2.1
Dam Monolith	1.8

4) Dewatered Case & Pseudo-Static Earthquake

Landside Monolith (L-4 through L-7)...............1.8

Riverside Monolith (R-36).........................1.8

These factors of safety have been computed assuming that a horizontal stress-relief crack exists below the base of each of the monoliths. The shear strength of this discontinuity is based on a combination of mineral friction and asperity, neglecting cohesion. Our selected peak friction angle is 50 degrees, and published data from in situ tests indicate shear strengths higher than the adopted value of 45 degrees are highly likely. The piezometers confirmed that such relief cracks, if present, do not transmit high hydrostatic pressures beneath the structure. Assuming this failure mode the resulting factors of safety are, in our opinion, conservative, and generally acceptable.

Overturning

The Overturning analysis utilized the method shown on Plate 40, and is consistent with the methods of analysis which define factor of safety as the ratio of available soil and rock strengths to the utilized soil and rock strengths. Development of the strength of the backfill and friction on the back of the wall was included in the analysis. The factor of safety as defined above the percent of base in compression are summarized below for the worst cases. Note the first entry, for example. While the percent of base in compression in only 49 percent, there is a factor of safety of 3.0 against exceeding the allowable foundation bearing pressure.

Condition	Factor of Safety Available Soil & Rock Strength / Utilized Soil & Rock Strength — Actual	Percent of Base In Compression — Actual
Normal Operation		
Landside Monolith (L-4 through L-7)	3.0	49
Riverside Monolith (R-36)	9.0	75
Dam Monolith	43.9	100
Dewatered Case		
Landside Monolith (L-4 through L-7)	2.7	38
Riverside Monolith (R-36)	6.8	50
Normal Operation & Pseudo-Static Earthquake		
Landside Monolith (L-4 - L-7)	1.7	23
Riverside Monolith (R-36)	4.8	38
Dam Monolith	48.3	100
Dewatered Case & Pseudo-Static Earthquake		
Landside Monolith (L-4 - L-7)	1.6	20
Riverside Monolith (R-36)	3.4	29

The percent of base in compression listed above is based on at-rest pressures. It is not possible to verify these calculations with field measurements, but in our opinion they are conservative.

The lowest factor of safety shown above, i.e., 1.6, was computed on landside lock Monoliths L-4 through L-7. It should be noted, Monoliths L-4 through L-7 have much smaller base widths than adjacent monoliths, refer to Plate 13, resulting in both smaller than acceptable percent of base in compression and factor of safety. In our opinion, factors of safety and percent of base in compression are adequate in all other instances for the lock and dam monoliths. However, it should be noted that riverside Monolith R-36 has 50 percent of its base in compression for the dewatered case with a factor of safety of 6.8 on bearing capacity. Since there is no uncertainty regarding loads in this case, and since performance has been favorable, we consider Monolith R-36 to be adequate.

It appears that there was a design change for Monoliths L-4 through L-7, probably to reduce the amount of rock excavation. However, the reason was not documented in the construction records. Rock anchors may have been installed in these monoliths to improve stability, but there appears to be no practical way to verify the number and capacity of such anchors, should they exist. It may be possible to locate such anchors using ground penetrating radar. This method has not been evaluated.

The stability of Monoliths L-4 through L-7 is particularly in question under earthquake loads. However, the performance history of the structure has been excellent and adds nothing to these concerns. The options at this point include 1) doing nothing and accepting the apparent small amount of base in contact, 2) conducting additional analyses, or 3) providing structural means of improving the stability. Additional analysis could include evaluating the side shear between monoliths as a group or evaluating the adhesion on the vertical concrete-rock interface behind the landside lock wall. These factors are normally not considered in the stability analysis of monoliths, but may make a significant contribution to stability. Structural means of improving stability could include rock anchors, shallow tiebacks to concrete deadmen, or underpinning the structure to increase the base width and embedment into rock.

RECOMMENDATIONS

Continued Monitoring

Monitoring of the instruments should be continued at monthly intervals for a period of at least several years in order to develop long term data on the response of the instruments over the various loading conditions and temperature extremes. As a minimum, the monitoring should continue until one year after the concrete repairs are completed. The instrument data should be plotted and reviewed quarterly by an engineer knowledgeable in dam design, instrumentation and the lock performance history. Limit values for each instrument should be established and the field personnel taking and reducing the data should notify the responsible engineer immediately if the limit values are exceeded.

Evaluation of Options

We recommend that parametric and cost studies be conducted to determine if additional studies, as discussed in Section 8.3, of the landside Monoliths L-4 through L-7 are likely to produce acceptable results. The relative costs of deep and shallow tieback schemes should also be determined. If these studies indicate that the cost of a tieback solution is relatively small, we would recommend the installation of a tieback anchor system on landside Monoliths L-4 through L-7; this may also be accomplished on several other monolith structures, depending on the factor of safety and the percentage of base in compression established by the COE as the governing criteria.

Fill Foundation Voids

Undercutting of the dam monoliths has been identified at three locations. The voids created at these locations should be filled with concrete to prevent further deterioration of the foundation under the dam.

Precautions

Sealing of the monolith construction joints could result in the blocking of natural drainage paths, resulting in higher hydrostatic pressures behind the landside lock monolith. If these repairs are made, it is our recommendation that one-way drains be installed through the lock monolith to drain the water level in the granular backfill.

Blasting to remove the deteriorated concrete could result in an increase in the lateral soil pressures acting on the landside monoliths. Measures should be taken to control blasting to minimize accelerations of the structure and backfill which could result in increased soil pressures.

Additional instrumentation should be installed and monitored prior to and during the blasting to measure wall movements and rotation during the blasting operations.

Methods of analyzing the stability of concrete structures on rock foundations are summarized in a report prepared by Shannon & Wilson, Inc. under REMR Program, Contract No. DACW 39-85-M-4183. The report is titled, "Review of Methods of Analyzing the Stability of Concrete Structures on Rock Foundations," and is included in these proceedings as Appendix A.

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TVA ASSESSMENT OF STABILITY OF CONCRETE STRUCTURES ON ROCK

Harold Buttrey

and

Hubert Deal, Jr.

Tennessee Valley Authority

1. The Tennessee Valley Authority (TVA) is an independent agency of the federal government created by ACT of Congress in May 1933. It is a corporation clothed with the power of government but possessed with the flexibility and initiative of a private enterprise. It is charged by the TVA Act with the duty of planning for the proper use, conservation and development of the natural resources of the Tennessee River drainage basin. TVA serves an area in the southeast made up of parts of seven states: Tennessee, Alabama, Mississippi, Kentucky, Virginia, North Carolina, and Georgia (Figure 1). The legislation which created TVA directs the agency to regulate the stream flow of the Tennessee River system in the operation of its dams and reservoirs primarily for controlling floods and promoting navigation and so far as may be consistent with these purposes to generate hydroelectric power. TVA also has the authority to take recreation into account in operating its reservoirs to the extent that it is not inconsistent with their operation for flood control, navigation, and electric power generation. TVA owns a total of 53 dams most of which have been designed, constructed, and are operated by the Agency. They include concrete gravity, earthfill, and rockfill dams or combinations of these types. They vary in height from a few feet for some of the earthfill dams to 480 feet for the Fontana Dam, a concrete gravity structure. Figure 2 provides some facts about major TVA dams and reservoirs.

2. TVA has essentially completed the development of the major hydro sites in the Tennessee River basin, and thus has not designed any new dams since the mid-1970s. Consequently, the hydro design unit has decreased considerably in number of personnel since the earlier days of TVA.

3. The present hydro effort at TVA consist of a continuing inspection and maintenance program and an evaluation and modification as needed of some 21 of our dams in compliance with the Federal Guidelines for Dam Safety. Therefore, our interest in their assessment of the stability of concrete structures on rock would be in support of these efforts.

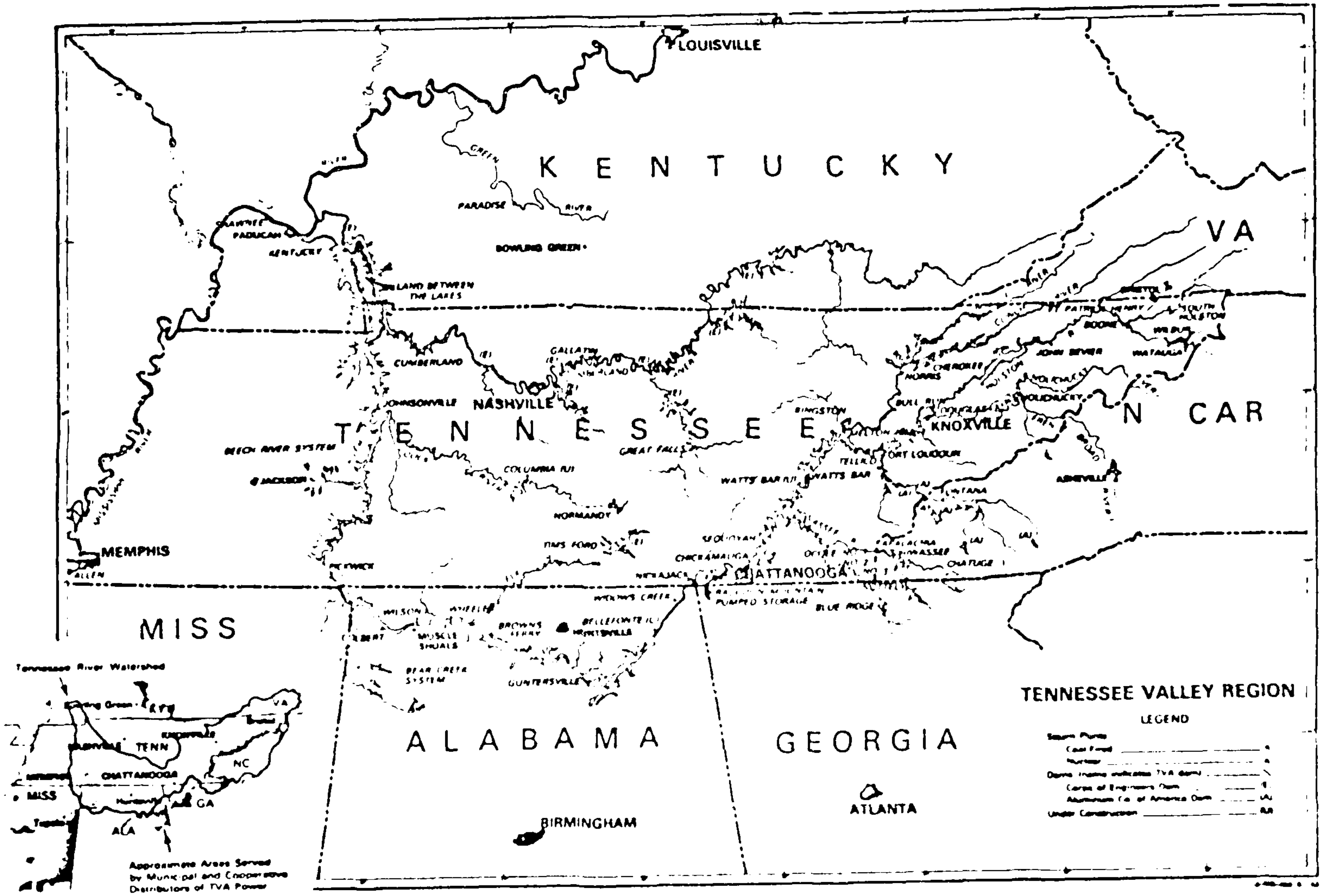


FIGURE I. – TVA SYSTEM

FACTS ABOUT MAJOR TVA DAMS AND RESERVOIRS

Main River Projects	Dam Max. Height (feet)	Dam Length Feet	Length of Lake (mi)	Area of Lake at Full Pool (acres)	Shore line at Full Pool (mi)	Lake Elevation (feet above sea level) Normal Min.	Top of Gates	Lake Volume (acre-feet) Top of Gates Elev.	Useful Controlled Storage	Construction Started	Closure of dam	Cost (millions)(e)	Ultimate Generating Capacity kW and No. of Units ()
Kentucky*	206	8,422	184	160,300	2,380	354	375	6,129,000	4,008,000	1938	1944	$119	175,000(5)
Pickwick Landing*	113	7,715	53	43,100	496	408	418	1,105,000	417,000	1934	1938	210	220,040(6)
Wilson (d)*	137	4,541	16	15,500	154	505	508	640,200	53,200	1918	1924	119	629,840(21)
Wheeler*	72	6,342	74	67,100	1,063	550	556	1,071,000	351,000	1933	1936	89	361,800(11)
Guntersville*	94	3,979	76	67,900	949	593	595	1,052,000	172,300	1935	1939	54	115,200
Nickajack (c)*	81	3,767	46	10,370	192	632	635	252,400	32,300	1964	1967	71	103,950(4)
Chickamauga*	129	5,800	59	35,400	810	675	685	739,000	347,000	1936	1940	40	120,000(4)
Watts Bar*	112	2,960	96	39,000	771	735	745	1,175,000	379,000	1939	1942	35	166,500
Fort Loudoun*	122	4,190	61	14,600	360	807	815	393,000	111,000	1940	1943	41	139,140
Tributary Projects													
Normandy	110	2,734	17	3,160	73	859	880	127,000	60,400	1972	1976	37.4	—
Columbia (b)	105	2,325	54	12,400	236	603	635	363,000	283,000	1973	(1986)	—	—
Tims Ford	175	1,484	34	10,600	246	865	895	608,000	282,600	1966	1970	52	45,000(1)
Apalachia	150	1,308	10	1,100	31	1,272	1,280	57,800	8,800	1941	1943	24	82,800(2)
Hiwassee	307	1,376	22	6,090	163	1,450	1,527	434,000	306,000	1936	1940	23	117,100(2)
Chatuge	144	2,850	13	7,050	132	1,905	1,928	240,500	122,500	1941	1942	9	10,000(1)
Ocoee No. 1 (d)	135	840	8	1,890	47	818	838	84,400	31,400	1910	1911	10	18,000(5)
Ocoee No. 2 (d)	30	450	—	—	—	—	1,115	—	—	1912	1913	29(f)	21,000(2)
Ocoee No. 3	110	612	7	480	24	1,413	1,435	3,300	3,080	1941	1942	9	28,800(1)
Blue Ridge (d)	167	1,000	11	3,290	65	1,590	1,691	195,900	183,900	1925	1930	5	20,000(1)
Nottely	184	2,300	20	4,180	106	1,735	1,780	174,300	117,100	1941	1942	8	15,000(1)
Melton Hill*	103	1,020	44	5,690	173	790	796	126,000	31,900	1960	1963	38	72,000(2)
Norris	265	1,860	129	34,200	800	960	1,034	2,552,000	1,922,000	1933	1936	32	100,800(2)
Tellico*	129	3,238	33	15,860	373	807	815	447,300	126,000	1967	1979	137	(a)
Fontana	480	2,365	29	10,640	248	1,580	1,710	1,443,000	946,000	1942	1944	77	238,500(3)
Douglas	202	1,705	43	30,400	555	940	1,002	1,475,000	1,252,000	1942	1943	45	120,600(4)
Cherokee	175	6,760	54	30,300	393	1,020	1,075	1,541,000	1,148,000	1940	1941	36	135,180(4)
Fort Patrick Henry	95	737	10	872	37	1,258	1,263	26,900	4,200	1951	1953	12	36,000(2)
Boone	160	1,532	33	4,310	130	1,330	1,385	193,500	148,500	1950	1952	27	75,000(3)
South Holston	285	1,600	24	7,580	168	1,675	1,742	764,000	438,000	1947	1950	31	35,000(1)
Watauga	318	900	16	6,430	106	1,915	1,975	677,000	354,000	1946	1948	32	57,600(2)
Great Falls (d) (on Cumberland Valley)	92	800	22	2,110	20	780	805	50,200	35,700	1915	1916	11	31,860(2)
Wilbur	77	375	2	72	4	1,645	1,650	715	327	—	1912	3	10,700
Nolichucky	94	482	—	383	26	1238.9	1240.9	2,070	—	—	1913	1.5	—
Pumped Storage													
Raccoon Mountain	230	8,500	—	528	—	1,530	—	38,180	36,340	1970	(1978)	(334)	1,530,000(4)
Totals				652,885	11,431			24,181,665	13,712,547				

*All main river dams and Melton Hill Dam are equipped with locks. A canal provides traffic access to Tellico Lake.

(a) Tellico project has no powerhouse. Streamflow through navigable channel to Fort Loudoun Reservoir will increase average annual energy through Fort Loudoun powerhouse by 200 million kWh.

(b) Under construction. Limited construction work at Columbia.

(c) Nickajack Dam replaced the old Hales Bar Dam 6 miles upstream.

(d) Acquired: Wilson by transfer from U.S. Corps of Engineers in 1933; Ocoee No. 1, Ocoee No. 2, Blue Ridge, and Great Falls by purchase from TEP Co. in 1939. Subsequent to acquisition, TVA heightened and installed additional units at Wilson.

(e) Cost of original construction plus major additions or rehabilitation.

(f) Includes cost of rehabilitation begun in January 1980.

May 1982, Information Office

FIGURE 2

4. We have particular interest in the change in the strength parameters with time in such areas as the rock-concrete contact, construction joints, and on any rock seams, bedding planes, etc. Methods to assess or estimate the change in strength parameters with time are also of primary interest.

5. Since the mid-1960s at TVA we have used the maximum probable flood (MPF) as the design flood and the probable maximum flood (PMF) to set dam freeboard and spillway containment wall heights. Also the operating basis earthquake and the maximum credible earthquake (MCE) have been developed for each site since that time.

TVA Assessment of Stability of Concrete Structures on Rock
Overturning Criteria

6. TVA's overturning criteria appears to be the same as that of most other organizations and has not changed to any significant degree in recent years. We are using the same criteria in our dam safety reevaluations that was used for most of our original designs, that is:

a. The resultant of all forces acting above any horizontal plane through a dam should fall within the middle third of that plane for normal loading conditions (Figure 3).

b. For the operating basis earthquake (OBE) combined with other forces, the resultant of all forces shall fall such that at least one-half of the base is in compression, assuming no tension (Figure 4).

c. For extreme loading conditions such as the probable maximum flood and the maximum credible earthquake the resultant of all forces shall fall within the base. If the resultant falls outside the base for the MCE and if other methods of analysis, such as the energy methods, indicate that the structure will not overturn during these extreme loadings, then the structure is considered adequate for these loadings.

Base Stress Criteria

7. Compression on the rock foundation shall not exceed 500 psi. Where foundation exploration and excavation reveal areas of weakness in the foundation, modification to the 500 psi may be required. Compression in the concrete shall not exceed 0.25 f'_c.

NORMAL LOADING - DAM

RESULTANT OF ALL LOADS (R) TO INTERSECT ANY BASE WITHIN MIDDLE THIRD OF THAT BASE

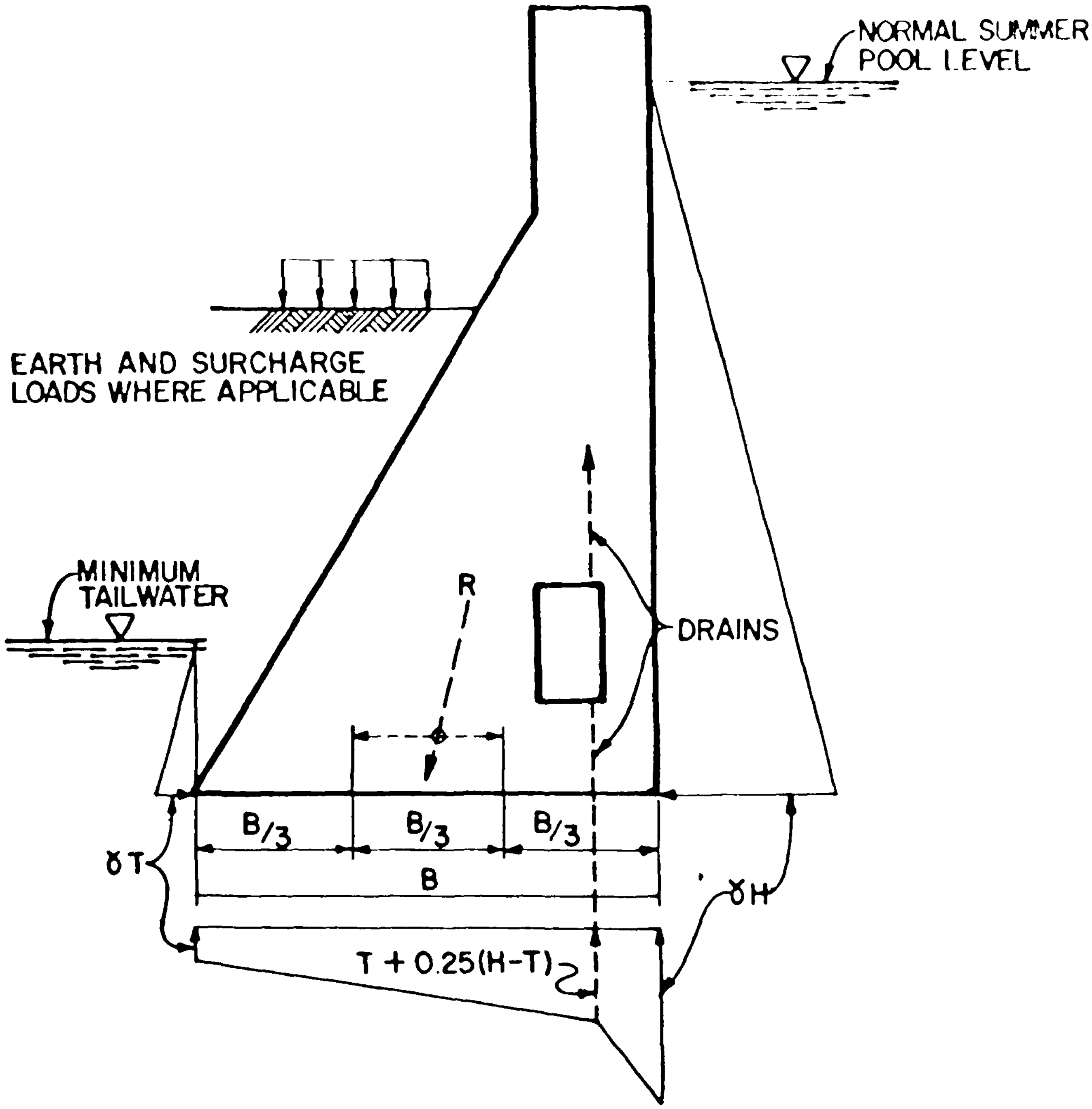


FIGURE 3

OPERATING BASIS EARTHQUAKE COMBINED WITH NORMAL LOADS

RESULTANT OF ALL LOADS TO INTERSECT ANY BASE WITHIN MIDDLE HALF OF BASE

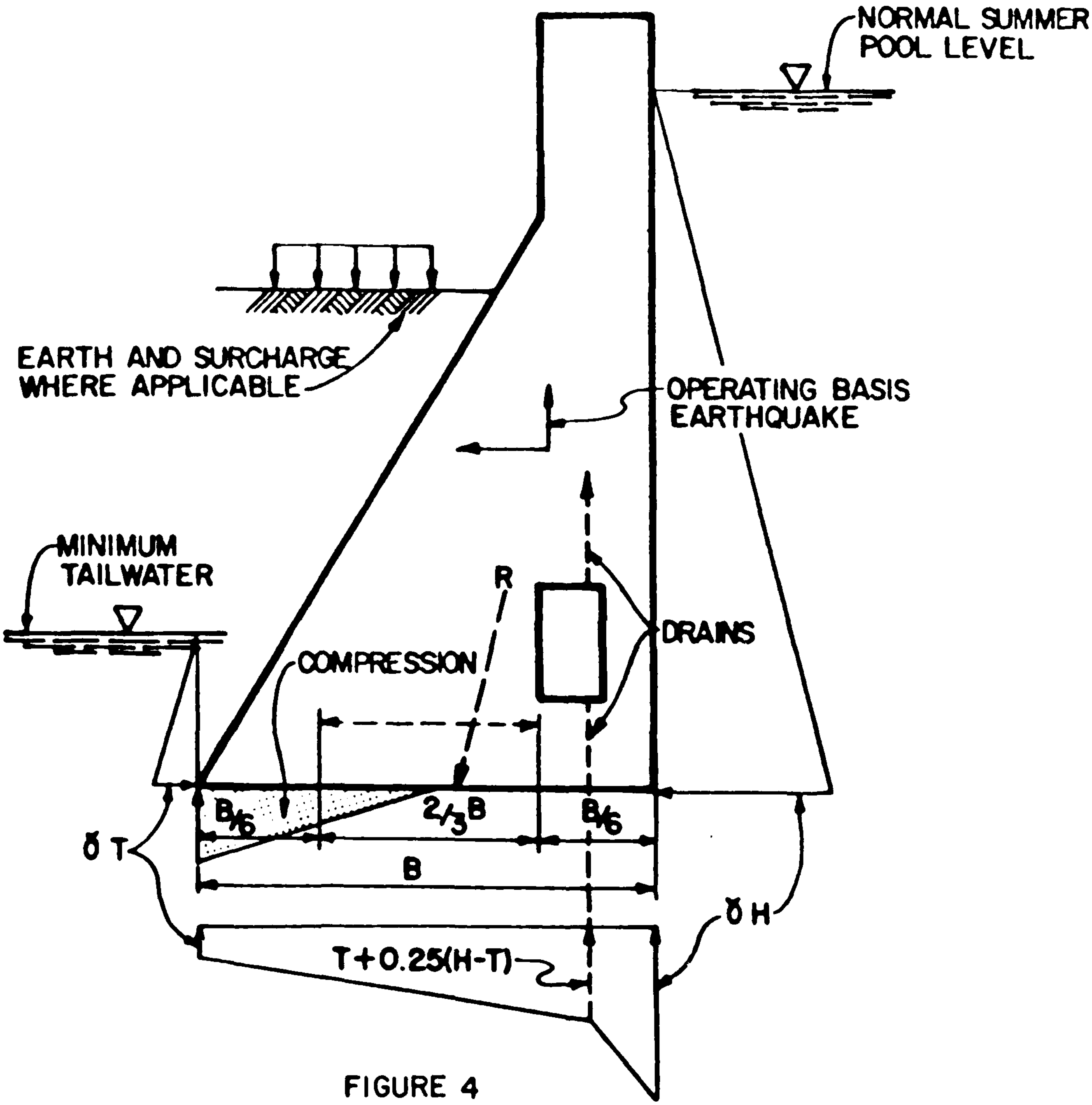


FIGURE 4

8. No tension is permitted on any base being analyzed except for the following loading conditions and is as given below:

a. Dead load only - 15 psi (Figure 5).

b. Horizontal loads acting in two directions, producing tension in one corner of the base - 15 psi (Figure 5).

c. For normal loadings combined with volume change or loads due to the maximum probable flood conditions stresses may be increased by 25 percent. For normal loadings combined with the design earthquake, stresses may be increased by 50 percent.

Uplift Criteria - Dams

9. For most of TVA's dams on rock foundation, means for measuring uplift pressures were installed during initial construction or have been added later. Most of TVA's dams have a line of foundation drains located a short distance downstream of the upstream face. In the earlier days of TVA various assumptions were used for uplift, such as uplift being from headwater at the upstream edge of the base being analyzed to tailwater at the downstream edge of the base acting over two-thirds of the base area. This assumption was modified on occasion for what was considered to be site-specific conditions. Monitoring of the uplift measuring devices and foundation drains led to uplift assumptions we use today and have used for several years.

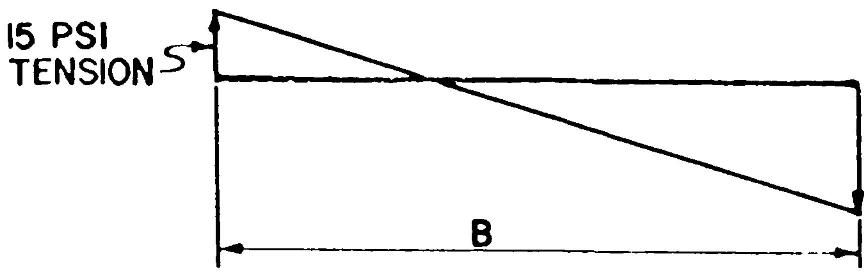
10. These assumptions are:

a. Uplift acts over 100 percent of base area.

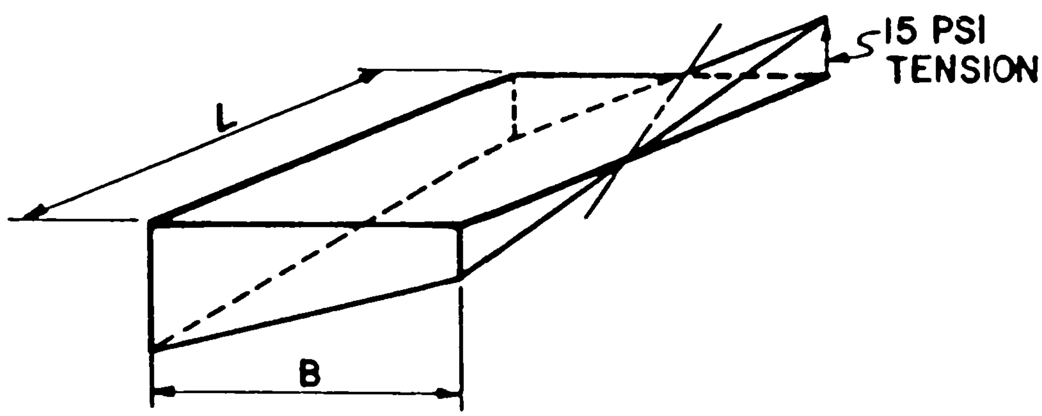
b. When the base being analyzed is below minimum tailwater elevation, the intensities of pressure for any plane shall be assumed to be equal to full reservoir head (H) at the upstream face, tailwater (T) at the downstream face, and T + 0.25 (H-T) at the line of drains.

c. When the base being analyzed is above minimum tailwater elevation and where the assumed plane of analysis is all or partially in rock, intensities shall be assumed to be equal to H at the upstream face, zero at the downstream face, and H/2 at the line of drains. Where the plane of analysis is through the concrete, intensities shall be in b, above.

ALLOWABLE BASE TENSION



DEAD LOAD CASE



HORIZONTAL LOADS ACTING IN MORE THAN ONE DIRECTION – PRODUCING TENSION IN ONE CORNER OF BASE

FIGURE 5

d. Where there are no drains, uplift is considered to vary in a straight line from headwater at the upstream face of the dam to tailwater at the downstream face.

Uplift Criteria - Locks

11. Where a lock wall is subjected to a differential head, the higher head is designated as H and the lower head as T (Figure 6.). The intensity of uplift on any base shall be assumed equal to T + 2/3 (H-T) at the higher head side and equal to T at the lower head side. Where a plane cuts through a lock wall culvert with a head H in the culvert, the intensity shall be taken equal to full head H from the inside face of the lock wall and extending across the culvert opening. From this point, the intensity varies uniformly to tailwater.
12. For blocks serving as part of the dam, the uplift intensity is assumed equal to H at the higher head side and equal to T at the lower head side.
13. Uplift pressure on navigation locks is assumed to act over 100 percent at the base area.

Uplifting Criteria

14. An extensive geologic exploration program was carried out on the foundation of all dams TVA has designed and constructed on a rock foundation. Even though this program identified weaknesses in the rock such as weathered bedding planes, seams, etc., that could affect stability analysis, in only a few cases has TVA attempted to take core samples and test these weaknesses to define strength parameters. As such, the parameters used for intact rock are considered to be conservative and in reality in most cases have been overly conservative. However, for probable weak seams the assumptions may not have always been conservative.
15. TVA still uses the shear friction method for determining resistance to sliding for structures on a rock foundation. We are familiar with the limit equilibrium method but have not used it to the extent as outlined in the Corps publications. Unless the exploration program indicates a need to establish more precise values, we have used 0.65 for tan ϕ and 250 psi for the unit shear strength in the rock foundation and at the rock-concrete contact and 400 psi for concrete. We require the factor of safety to be at least four

UPLIFT - LOCK WALLS

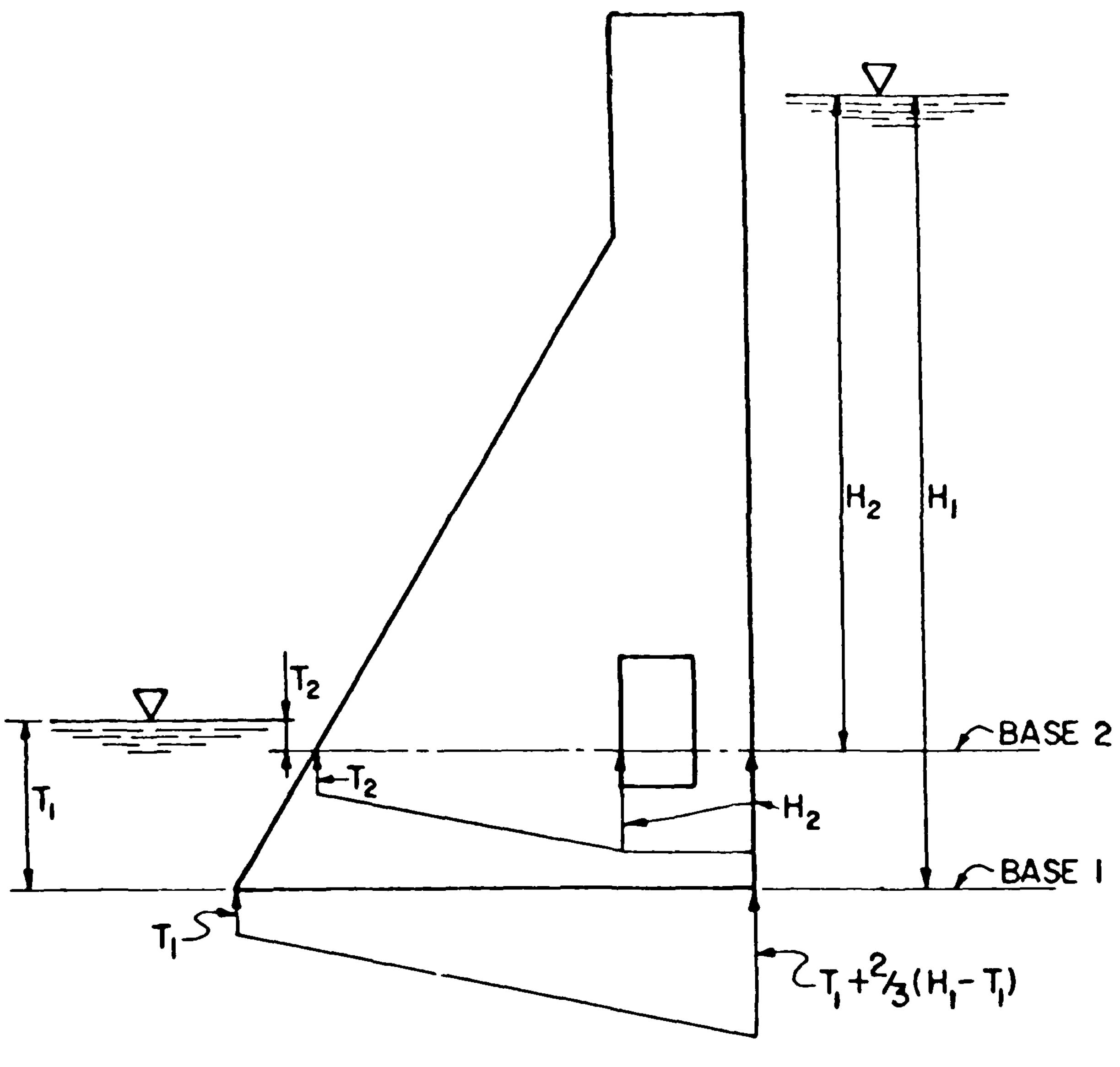


FIGURE 6

using this method. For the dams we have designed since the early 1960s, we have taken into account any passive wedge where the exploration indicates the quality of rock appears to support this judgment. Where used, this has been a judgment decision not based on test. We have not attempted to anchor an otherwise questionable wedge to sounder rock in order to provide for passive support.

16. TVA has the facilities to extract and test rock cores to determine strength parameters of any weak areas, and in the past has tested NX-size cores taken from shale layers and 3.0-foot-diameter calyx cores drilled from the foundation of a damsite to check the strength of weak seams. In our safety analysis of our dams, if it proved economically advantageous to go to the expense of testing weak areas of a foundation and analyze by the limit equilibrium method, and allow a lower factor of safety, we would do so.

Special Case - The Tims Ford Dam

17. The Tims Ford Dam is a rockfill dam with a sloping impervious core that was designed and constructed by TVA in the mid-1960s in Middle Tennessee. It is approximately 1,580 feet long and 175 feet high. Original plans called for a conventional concrete gravity dam, ogee spillway, intake, and powerhouse. After construction began, a program of foundation exploration consisting of extensive core hole drilling and several 36-inch-diameter calyx drill core holes was initiated. It was found that what initially appeared to be bedding planes between shale and limestone layers were continuous weak seams of decomposed shale (Figure 7). The presence of the weak seams and the fact that they daylighted downstream gave concern as to whether they would have sufficient shear strength to resist sliding in the foundation when the concrete dam was loaded. A program was developed and initiated to perform shear strength tests of the weak seams by removing and testing 36-inch-diameter cores drilled from the foundation. As a result of these tests it was determined that the shear strength of the foundation was too low to provide an adequate factor of safety against sliding to ensure the safety of the concrete structures. Therefore, the decision was made to abandon plans for a concrete gravity dam and to provide a compacted rockfill dam with sloping impervious core instead.

18. The results of the testing program led to the conclusion that the stability analysis of the dam should be done for two sets of strength parameters for the weak seams. They were ϕ = 20 degrees and cohesion of 600 psi and ϕ = 25 degrees and cohesion of 0 psi. A ϕ of 20 and cohesion of 600 psi proved to be the controlling strength.

BUREAU OF RECLAMATION CURRENT PRACTICES
OF CONCRETE STRUCTURES ON ROCK FOUNDATIONS

AD P005684

Howard Boggs
Bureau of Reclamation

Introduction

1. The Bureau is currently in the process of rewriting criteria and preparing design standards and guidelines for concrete dams. The large thick books, Design of Arch Dams, Design of Gravity Dams, and Design of Small Dams, are to become Design Standards. We are going to put out a standard for everything.

2. The Bureau and others have similar criteria for retaining walls, that is, a safety factor of 1-1/2. The training walls and channel floor are all self-contained. So far as concrete structures are concerned, the following comments will address mainly concrete dams. The Bureau has about 55 concrete dams of which 28 are arch.

3. For concrete dams, we have basically four types of structures, arch or gravity, and new or existing structures. The criteria differs somewhat for each one. An extensive discussion will not be presented about United States Bureau of Reclamation (USBR) arch dams except to show their influence on the design and analyses of gravity dams using those concepts. From the structural analysis of arch dams, the designer becomes very cognizant of three-dimensional, homogeneous, isotropic structures. This awareness is carried over to the gravity dam design. Concrete blocks are all uniformly cooled from base to crest, keys are formed during construction between the contraction joints, and ultimately all of the contraction joints are grouted. A similar concept applies to gravity dams also. With gravity dams, the primary interest is in sliding using the sliding friction method which is the resistance divided by the driving force to define the safety factor. In arch dam analysis, we are primarily interested in stresses.

4. Safety factors for both stress and sliding stability arch and gravity structures are: (a) 3.0 for the usual loading combination which includes

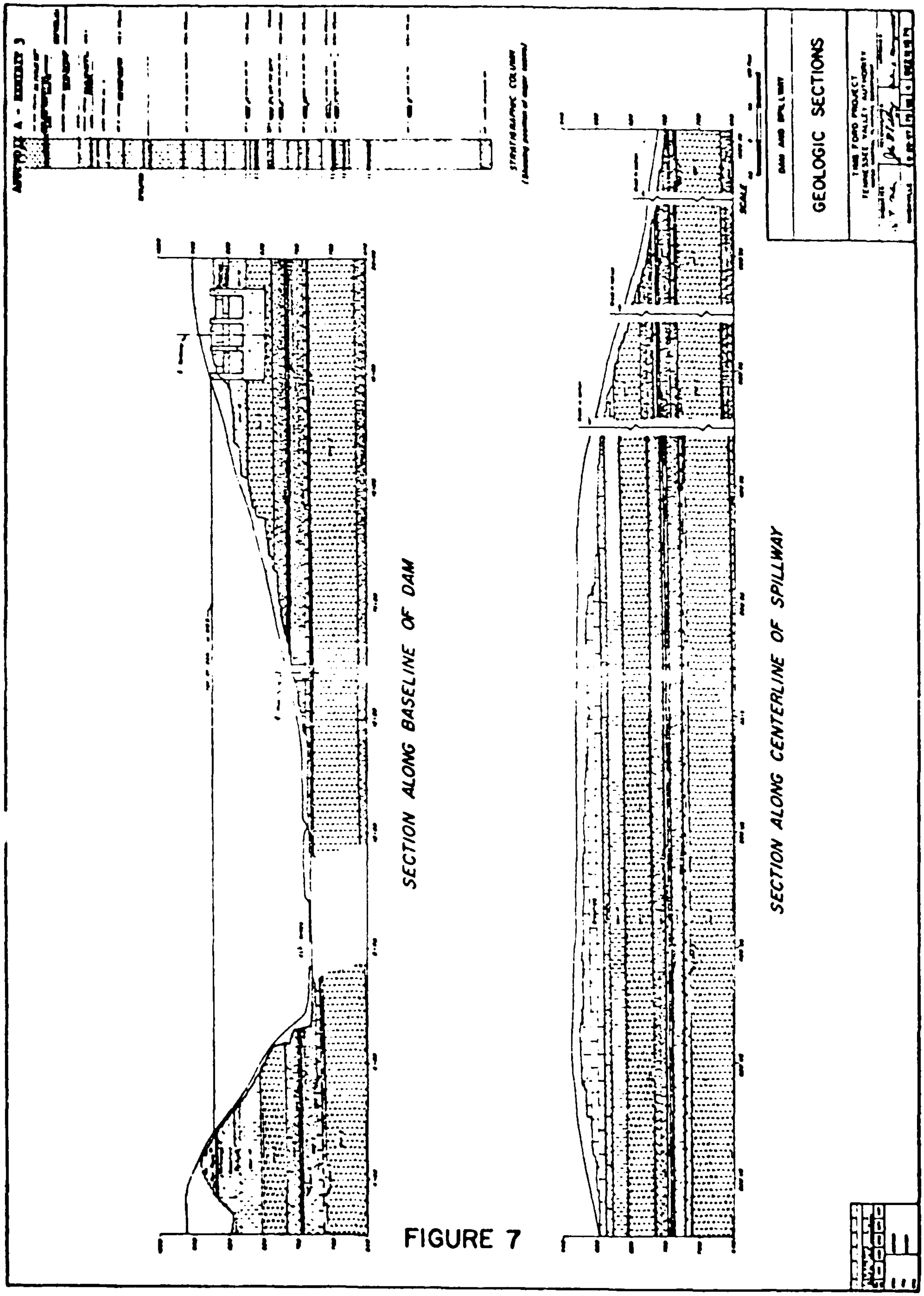

SECTION ALONG BASELINE OF DAM
SECTION ALONG CENTERLINE OF SPILLWAY
STRATIGRAPHIC COLUMN
DAM AND SPILLWAY
GEOLOGIC SECTIONS
TIMS FORD PROJECT
TENNESSEE VALLEY AUTHORITY

FIGURE 7

mainly the reservoir operation with gravity and temperature, and which occurs about 96 percent of the time, (b) a safety factor of 2.0 for the unusual or flood condition with gravity and temperature which occurs about 4 percent of the time, and (3) a safety factor of greater than 1 for the earthquake or extreme condition which is the rest of the time. For allowable stresses, the maximum stress for the usual condition is 1,500 lb/in^2, 2,250 lb/in^2 for the flood loading and less than the concrete strength for the extreme loading.

5. Currently in the USBR, foundation analyses include the sliding friction (or the Coulomb equation) and applicability of the passive wedge analyses. Suggested safety factors are 4 for the usual condition, 2.7 for the unusual, and 1.3 for the extreme condition. Foundation treatment is the usual type; consolidation grouting, curtain grouting, and drainage systems. Special conditions are addressed such as in the case of sedimentary beds dipping upstream or where a horizontal keyway is required. In general, treatment is similar to that provided by other agencies.

Loads

6. Usual. Individual loads applied to the dam are the usual reservoir operation which is either normal water or the minimum, gravity, temperature, silt and ice. Temperature for an arch dam is extremely important; on smaller dams it is more important than the water load. For this reason, very extensive thermal and stress analyses are made to determine the effects. Ice is applied as a pressure of 5 kips per foot of ice thickness per linear foot of structure. Normally, the ice load is not a problem. The size of concrete structures 50 feet or greater tends to mitigate the effects of the ice load. Silt load is considered as an increase of about 22-1/2 pounds per cubic foot on the water load density.

7. Unusual. The flood load in any analyses is the Probable Maximum Flood (PMF) condition which is the maximum height of the dam for new structures. A continual re-evaluation of the flood condition results shows that some structures may be overtopped. Overtopping of concrete dams is a concern, but not catastrophic at this point in time. In 1967, an arch dam, Gibson Dam in Montana, overtopped for about 3 days without structural damage except for inundation of the abutments and the service road being washed out. Later,

the abutment, for some distance downstream, was covered with 5 feet of concrete in lieu of increasing the spillway size.

8. Extreme. The USBR method of earthquake analysis is to perform a response history analysis on all concrete dam analyses when required. Magnitudes and corresponding epicentral distances for historical events within a 200-km radius are tabulated. From this list, the most severe Maximum Credible Earthquake (MCE) is selected and a smooth response spectra is developed based on known accelerograms. Digitized accelerograms are developed, scaled appropriately for the direction, and applied to the particular type of structure, whether it is a three-dimensional arch or a two-dimensional gravity dam. The three directions are upstream-downstream, cross-canyon, and vertical.

Methods of Analysis

9. Stress analyses for the gravity dam or the arch dam use the ordinary beam theory or the finite element method. Also used for the arch dam is the trial load method or the computerized version called Arch Dam Stress Analyses System (ADSAS) analyses. The finite element method currently in use is SAP-IV, linear elastic method. Three-dimensional finite element method analyses assume conventional concrete dams are monolithic. Proper evaluation of results from earthquake analyses may require a somewhat subjective judgment. The ADINA computer program, from M.I.T., is being modified in an attempt to realistically assess contraction joint response during earthquake. Currently, Chopra's EADHI and EAGD-84 computer programs are being used for our gravity dam analyses during earthquakes.

Foundation

10. In the foundation, primary analyses are the shear friction factor of safety, passive wedge analyses, and some of the progressive failure modes where appropriate. This latter method is used in conjunction with the finite element method analysis.

Instrumentation

11. The correlation between design and prototype behavior is determined from structural analyses and instrumentation. Many instruments are installed in each dam during construction. On a routine basis, measurements are recorded, plotted, and evaluated. This procedure is very time consuming, but is the only sure method of back-checking the original design assumptions. Some of the instruments installed in our dams are strain meters, stress meters, joint meters, thermometers, and weirs. In the foundation, instrumentation includes piezometers, MPBX's in at least two directions along the line transverse to the axis, and pressure gauges. Plumb lines are a very reliable and informative measure of the deflection of the structure throughout the year, and repeatability of plumb line measurements is extremely valuable and accurate in developing trends or noting anomalies.

12. In high double curvature arch dams, where the vertical drop down the crown cantilever to the base is not totally within the sections, a substitute for the plumbline using tiltmeters is being installed in Morrow Point Dam on the Gunnison River, Colorado. Computed deflections from the tilt and the subsequent double integration in the vertical direction from base to the crest will be compared with measured deflections from a juxtaposed plumb line to determine if another measuring device is applicable and accurate. Surface measurements include the customary surveying methods, Electronic Distance Measurement (EDM), and collimation. Accuracy of collimation measurements sometimes become marginal because of the distance across the canyon. This fact serves to emphasize the need for redundancy in structural behavior measurements.

13. Extensometers anchored in the dam and the foundation have proven to be very valuable in measuring the reaction of the structure to the applied loads. At Glen Canyon Dam, a 700-foot-high structure on the Colorado River, extensometer measurements showed that after about 7 years that structure and the reservoir settled down before the system began to repeat the oscillations due to the temperature and the water fluctuation. Extensometers in the foundation in Pueblo Dam, a 180-foot-high massive head buttress dam in Colorado, showed the crest to be deflecting upstream. Our instrumentation group rechecked the measurements and inquired if new personnel had been assigned the job of making

measurements. After confirming the accuracy of the original data, engineers from the design and instrumentation sections evaluated the collimation data along the crest, the extensometer data from the foundation, and other measurements before deciding that the dam really was moving upstream at the crest. This conclusion was confirmed with a two-dimensional finite element analysis of the massive head buttress, the foundation under the dam, and the reservoir. Results showed that when the foundation deflected from the reservoir load, it rotated the dam upstream.

14. Weirs installed in the foundation gallery to measure seepage proved to be a very unique measuring device in another aspect. Periodically, in Hungry Horse Dam in Montana, anomalous increases in the flow on one side of the dam were recorded which were completely off the scale, but in a year or two they would return to normal. Back-analysis disclosed that there were several simultaneous earthquakes in the area. In addition, geological and construction records showed the dam was built across a fault. Thus, whenever an earthquake within a hundred miles of the dam occurred, the drain flows would temporarily and significantly increase before resuming normality.

15. Uplift is a hydrostatic condition affecting all concrete structures to some degree. Based on calculations and measurements, uplift in thin arch dams is not a major concern. Stress analyses including uplift have shown that most any variation in uplift assumption is not going to significantly change the stress or stability of the dam. However, within a gravity dam, uplift is important. Uplift varies linearly from reservoir pressure to one-third the pressure difference between reservoir and tailwater at the line of drains to tailwater pressures, presuming the drains are working.

16. To substantiate stability and track stability including uplift, operable drains and companion measurable weir flows are compared with pressure gauge readings, such as, if the pressure goes up and the flow goes down, a problem might exist. Four or five pressure gauges are generally equally spaced along a radial line, i.e., radial to the upstream face in the transverse direction. Three or more lines of pressure gauges are located longitudinally across the dam, whether it is arch or gravity.

Maintenance

17. To assure that structures are operating as designed or in an acceptable fashion, two investigations are used: Review of Maintenance (ROM) and Safety Evaluation of Existing Dams (SEED). Since the late 1940's, a ROM program has been organized in which the staff from our Denver office, regional offices, or project offices would go out every 2, 4, or 6 years routinely and examine the structures. These inspections primarily addressed chipped paint, grass growing on the abutment, seepage, grass on the downstream face of the dam, or any other such thing that would cosmetically be unacceptable. These reports have become very valuable in evaluating the history of the structure. Always during the inspection, the staff would take many pictures. Consequently, when a problem appears to be developing, we peruse the photographs searching for a sequence of what has happened in or on the structure during the last 40 years. ROM reports have become very valuable in the sense that sometimes from historical reports the design conditions can be determined. These documents coupled with the instrumentation and ROM records are about the only way that some structures can be evaluated. In evaluating records dating back to the early 1900's, specifications (usually with only about four drawings) are very limited as far as trying to reconstruct early structural behavior, especially during construction and early operation of the structure. Therefore, all documentation is very valuable in assessing the current structural safety.

18. In the SEED program the structure is inspected by a team of qualified people, i.e., civil and mechanical engineers and a geologist, who also review and evaluate the design data, construction records, instrumentation records, or other data. Subsequently, the team recommends additional studies. And then once those recommendations are made, other sections of the organization respond to it. The SEED reports usually recommend a state-of-the-art earthquake analysis. From these recommendations and limited data, a structural analysis is made of the dam and foundation. If something is amiss, such as a tensile stress that exceeds the probable tensile strength, more data may be needed either from another field trip by the analyst, or from in situ or laboratory tests. A second inspection may be necessary because of distress potential from the structural analysis. Old structures may have cracks from either structural or material deterioration, and pose problems with stability.

Additional data are required prior to modification designs under Safety of Dams (SOD) authorization.

Concerns about the Stability of Concrete Dams

19. What does stability really mean? How can we accurately measure stability? Can we develop a process relating analytical methods with structural model methods or prototype methods? The finite element method of analysis is a very efficient and accurate method; however, it is still an analytical method and results should be verified with other types of measurements.

20. If we have structural damage from an earthquake, or if we have a flood that exceeds the capacity of the reservoir, and we anticipate overtopping, what is the effect from overtopping or earthquake damage regarding the scour and potential instability?

Current USBR Concrete Dam Research

21. Current research is limited on concrete dams. One project we have going on now is the development of a contraction joint finite element code for the ADINA computer program. This joint element is a nonlinear element that acts during an earthquake or during extreme temperature loading to accurately model the response and redistribution of loads on an arch dam or gravity dam.

22. The application of fracture mechanics to concrete dams is being evaluated. Existing USBR methods of analyses for cracking of gravity dams or arch dams are conservative and should be confirmed or modified.

23. Another ongoing research project is intended to determine the effectiveness of foundation grouting during construction. Unknowns to be determined are (a) how long does it last?; (b) how efficient is it?; (c) must the foundation be regrouted?; and (d) does it really do the job? To evaluate these parameters, acoustical measurements and other downhole measurements are performed before and after grouting. If you know of any answers to these questions, the Bureau would welcome assistance and test results.

FEDERAL ENERGY REGULATORY COMMISSION
DAM SAFETY PROGRAM

Jerry Foster
Federal Energy Regulatory Commission

Introduction

1. The Federal Energy Regulatory Commission (FERC), Office of Hydropower Licensing, is responsible for the regulation of nonfederal hydroelectric power projects. All hydroelectric facilities not owned and operated by a federal agency, such as the Corps of Engineers, TVA or USBR, must obtain an operating license from FERC. The Federal Power Act, first enacted in 1920, gives FERC broad powers to insure public safety and proper utilization of water resources for hydroelectric power generation. FERC currently has jurisdiction over approximately 2200 hydroelectric projects involving approximately 790 dams over 35 feet in height and, therefore, has a keen interest in the safety of existing dams.

2. The dam safety program at FERC is administered in two ways. First, all projects for which an application for license is received or for which a major change in development is proposed, must be certified as "safe and adequate" by Design Review Branch (DRB) Engineers. Each project is subjected to a review of the hydrologic, hydraulic, geotechnical and structural adequacy of its major features. Prior to licensing, all projects must be shown to be safe, either by staff or applicant studies, or a plan for demonstrating the safety of the structures must be developed as a condition of licensing.

3. After licensing, each project is inspected annually by FERC Regional Office inspectors, and once every 5 years by independent consultants, under Part 12 of the Power Act. Those inspections are conducted in order to insure projects are being properly maintained, that no unauthorized modifications have been made, and that the project is being operated efficiently and safely.

4. The dam safety program has been successfully in identifying potentially unsafe and hazardous dams. FERC has required over 50 dams to undergo varying degrees of rehabilitation in the past 5 years. Table 1 shows a sample of the projects, and type of rehabilitation required, which have been upgraded under the dam safety program.

24. USBR publications available are "Design Criteria of the Concrete Arch and Gravity Dams," "Design of Gravity Dams," "Design of Arch Dams," and "Design of Small Dams."

TABLE 1

FEDERAL ENERGY REGULATORY COMMISSION

PROJECT NAME	STATE	FERC NO	HEIGHT (FT)	HAZARD CATEGORY HYDROL	HAZARD CATEGORY SEISM	DAM TYPE 2/	DEFC TYPE 1/	REMEDIAL ACTION REQUIRED	DATE CONSTR	DATE UPGRA 3/
BAGNELL	MO	459	139	HIGH	1	1	2	POST-TENSIONED	1931	1982
BARKER	CO	1005	175	HIGH	1	1	2	POST,GROUT,DRAIN	1910	1984
BARTLETTS FERRY										
BLACK BROOK	WI	2894	25	LOW	1	1	5	SHEET PILE,APRON	1917	UNDW
BOARDMAN	MI	2979	54	HIGH	1	2	5	FILTERS,DRAINS		1984
BULL RUN NO 1	OR	2821	194	HIGH	2	5	2	ADD DRAINS	1929	1978
CADYVILLE	NY	2738	50	HIGH		1	2	POST-TENSIONED	1922	UNDW
CALDPEN FALLS	WI	2525	39	HIGH	1	1	1	POST-TENSIONED	1924	UNDW
CARRY FALLS	NY	2060	76	HIGH		2	4	DRAINS ADDED	1953	1984
CHECAH	NC	2169	222	HIGH				DRAINAGE ADDED	1919	1984
CLAYTOR	VA	739	120	HIGH	2	1	1	THRUST BLK,POST	1939	1985
CONOWINGO	MD	405	106	HIGH	1	1	1	POST-TENSIONED	1928	1978
CUSHMAN NO 1	WA	460	260	HIGH		4,2,1	1	POST,RES. LOWERED	1926	1976
DELTA	NY	7610	100	HIGH		1	2,1,4	POST,GUNITE	1913	1985
ELWHA	WA	2683	100	HIGH	3	1	1,5	POST,INSTRU	1912	1980
CLINES CANYON	WA	588	195	HIGH	3	1,2	1,2	POST-TENSIONED	1927	1977
GOAT ROCK	GA	2177	68	HIGH		1	2	POST-TENSIONED	1912	UNDW
GRANDFATHER FALLS	WI	1966	32	HIGH	1	1	4	NEW SPILLWAY	1906	1983
HARDEY	MI	2452	100	HIGH	1	1	5	FOUND. GROUT	1931	1984
HARRIET LAKE	OR	135	68	LOW	2	4	4	GROUTED	1923	1984
HARRIMAN	MA	2323	216	HIGH	2				1925	
HAUSER	MT	2188	125	HIGH		1	2	POST-TENSIONED	1911	1980
HENSHAW	CA	176	155	HIGH		2,4	1,2	BERM ADDED,SPILLW IN	1922	
HINKLEY	NY	3211	83	HIGH	1	1,2		POST,RESURF FACE	1914	1984
KENT FALLS	NY	2738	58	HIGH		1	2	POST-TENSIONED	1928	UNDW
LINNVILLE	NC	2232	140	HIGH		2	4	DRAINS	1919	1983
LOWER BARKER	WA	2150	277	HIGH	2	4	5	GROUTED	1927	1983
MAMMOTH POOL	CA	2085	411	HIGH	3	2	1	NEW GATE STRUCTURE	1960	1985
MARTIN	AL	349	161	HIGH	1	1	2	POST-TENSIONED	1926	1984
MILL C	NY	2738	42	HIGH		1	2	POST-TENSIONED	1922	UNDW
MILLINOCKET LAKE	ME	2458	20	HIGH		2	1	DAM RAISED		UNDW
MILLTOWN	MT	2453	52	LOW		1	5		1907	1984
MORGAN FALLS	GA	2237	56	HIGH		1	2	POST-TENSIONED	1904	UNDW
MORRISVILLE	VT	2629	37	SIGN	2			POST-TENSIONED	1924	
MOSHIER	NY	2645	93	HIGH		2	5	DRAINS CLEANED	1929	UNDW
PETENWELL	WI	1984	40	SIGN	1	2	4	INSTALL FILTERS	1949	1985
PINEY	PA	309	125	HIGH	1	1	2,1	POST-TENSIONED	1926	1983
PINOPOLIS	SC	199	77	HIGH	3		3		1942	UNDW
POINT A	AL	2586	35	HIGH		2	2	FILL ADDED,SLOPES FL	1924	1982
POTATO RAPIDS	WI	2560	19	SIGN	1	1	5	UNDERPIN.,GROUT	1921	1983
RAPIDAN	MN	3071	73	HIGH	1	6	2	POST-TENSIONED	1910	1984
ROCKY REACH	WA	2145	118	HIGH	2	1	2	POST-TENSIONED	1962	UNDW
SALMON CREEK	AK	2307	170	HIGH		4	2	LOWER RESERVOIR	1916	1983
SALUDA	SC	2406	55	HIGH	3				1905	
SANTEE	SC	199	37	HIGH	3		3		1950	UNDW
SCOTT	CA	77	135	HIGH		1	2	ADD BUTTRESS	1921	1984
SOFT MAPLE	NY	2645	140	HIGH		2	5	DRAINS ,FILTERS	1925	UNDW
STEWARTS BRIDGE	NY	2047	112	HIGH		2	5	FILTER ADDED	1952	UNDW
THOMSON	MN	2360	16	HIGH	1	1	2	POST-TENSIONED	1907	1982
TRENTON FALLS	NY	2701	60	SIGN					1901	
TWIN BRANCH	IN	2570	37	HIGH	1	1	5	GROUT	1903	1983
VICTORIA	MI	1864	115	LOW	1	6	4	REPAIR BUTTRESSES	1931	1985
WALLENPAUPACK	PA	487	66	HIGH	1	1		POST-TENSIONED	1926	UNDW
WALTER BOULDIN	AL	2146	165	HIGH	1	2	3,5	BERM ADDED	1967	1984
WELLS	WA	2149	145	HIGH	2	1	2	DRAINS AND MONITORIN	1968	1984
WISCONSIN RAPIDS	WI	2256	31	HIGH	1	1		GROUT MASONRY	1903	1984

NOTES:

1. Deficiency Types
   1-Insufficient spillway capacity
   2-Unstable structure
   3-Unstable foundation
   4-Structural deterioration
   5-Foundation deterioration
   6-Design deficiency
   7-Dam failure

2. Dam Types
   1-Concrete gravity-straight
   2-Embankment-earth
   3-Embankment-rock
   4-Concrete-arch
   5-Concrete-gravity curved
   6-Concrete-buttress

3. UNDW-construction underway

Safety of Existing Dams

5. Dams found to be unsafe usually fall into two general categories: either the safety criteria or analysis procedures (hydrologic or structural) have changed since the dam was designed and constructed; or deterioration of the structure (or foundation) required a reassessment of its safety based on current site conditions.

6. A large number of existing concrete gravity dams under FERC's jurisdiction were designed and built prior to the 1930's. As these projects apply for relicensing or amendment, they must be re-evaluated under current hydrologic and structural criteria. The most commonly noted deficiency of these dams is an inadequately sized spillway, resulting in either theoretical overtopping of the dam or/and increased loading on the structure during a Probable Maximum Flood (PMF) event. Other problems noted are: changed loading due to increased uplift pressures, foundation leakage or deterioration, cracked or deteriorated concrete, and a change in downstream development which requires an increase in the hazard potential of the project.

7. The above factors result in many existing dams not meeting current dam safety criteria. This presents the problem of determining which dams, many of which have performed satisfactory for over 50 years, should be required to be upgraded to present-day standards. Not only must the decision be made as to which dams should be rehabilitated, but it also must be decided to what degree of safety the structure must be upgraded. Rehabilitation of an existing structure is an expensive and difficult undertaking. The catastrophic consequences of a dam failure makes these important decisions more difficult.

8. At FERC, the DRB Staff uses a three-step procedure to identify unsafe dams which need rehabilitation. First a hydrologic analysis of the project is made, based upon its hazard rating, and a spillway design flood is established. For high-hazard projects, the Spillway Design Flood (SDF) is the PMF. The SDF is routed through the project to determine the loading on the structure and ability of the spillway to handle the flow. A stability analysis of the major project features is then conducted for a range of loadings up to the SDF and for earthquake loading. If the stability analysis indicates a safe dam, then the process is complete; however, if an unsafe dam is indicated, a decision must be made whether or not and to what extent the project must be rehabilitated. For unstable dams the third step, a safety evaluation, is

conducted which considers the impact of failure on downstream river stages at the time of failure. This requires a dambreak analysis and flood routing through the critical reaches of the downstream river basin. If the river stages downstream are not significantly affected by a dam failure, then no remedial action is required. If the converse is true, then a plan for the project must be submitted and approved by FERC Engineers.

Stability Analysis

9. The criteria and procedures used to conduct stability analyses of concrete gravity dams on rock at FERC are a combination of Corps of Engineers and Bureau of Reclamation requirements, with some modifications made based on the FERC experience with existing dams.

10. The loading conditions used in the stability analysis of gravity dams are as follows:

a. Case I - Usual Loading Combination - Normal Operating Condition. The reservoir elevation is at the top of normal power pool, or the top of the closed spillway gates, whichever is greater. Minimum normal tailwater is used and ice pressure, if applicable, should be considered. Horizontal silt pressure should also be considered if applicable.

b. Case II - Unusual Loading Combination - Flood Discharge. The project flood which results in reservoir and tailwater elevations that exert the greatest head differential and difference in moments upon the structure should be used. This may result in the use of a flood of lesser magnitude than the Spillway Design Flood. Many overflow spillways will be submerged during periods of high discharge. Failure of the structure while submerged may be less critical, in terms of the flood wave released, than failure during a period when the tailwater is low. Tailwater pressure should be taken as full value for nonoverflow sections and 60 percent of full value for overflow sections, except that full value should be used for computation of the uplift.

c. Case III - Extreme Loading Combination - Normal Operating with Earthquake. The same loading as in Case I is used except that the inertial forces due to the earthquake acceleration of the dam and the increased hydrostatic forces due to the reservoir reaction on the dam are added.

11. The basic requirements for stability of a gravity dam for load Cases I and II are:

a. That it be safe against overturning at any horizontal plane within the dam, at the base or at any plane below the base. This requires that the allowable unit stresses established for the concrete and foundation materials not be exceeded. The allowable stresses should be determined by dividing the ultimate strengths of the materials by the appropriate safety factors.

b. That it be safe against sliding on any horizontal plane within the dam, on the foundation, or on any horizontal seam in the foundation. The ultimate value of cohesion required for stability should be solved for using the appropriate safety factors.

12. For load Case III the requirements for stability are:

a. For an earthquake loading using the seismic coefficient method, the basic requirements for stability under Case I and Case II loading apply.

b. For an earthquake loading using dynamic or pseudo-dynamic methods, the following criteria apply:

(1) The dam shall be capable of surviving a Maximum Possible Earthquake (MPE) without a failure of a type that would result in loss of life or significant damage to downstream property. Inelastic behavior with associated damage is permissible under the maximum possible earthquake for the site.

(2) The dam shall be capable of resisting an Operational Earthquake (OE) within the elastic range of the materials. An Operational Earthquake shall be defined as one which is likely to occur during the life of the project as determined through geologic and seismic studies.

13. The procedures used in the conventional rigid body analysis are basically those used by both the Corps and the Bureau, with the exception that FERC has adopted the Bureau's method of handling uplift. For reasons explained later herein, the Bureau's method for determining the initiation of interface cracking is considered more appropriate for existing dams.

Examples of Rehabilitated Concrete Gravity Dams

14. Following are three examples of projects which were found to be unstable and which were rehabilitated under FERC's dam safety program. These dams were selected because they demonstrate the types of problems common to many existing structures, and each also presented site specific problems requiring unusual solutions.

Elwha Project

15. The Elwha Dam, located on the Elwha River near Port Angeles Washington, was constructed in 1912. As the reservoir was raised, the project began experiencing a long history of stability problems. On 30 October 1912, the foundation under the nonoverflow portion of the dam failed, allowing the reservoir to be drained, flowing under the dam. An examination of the geologic cross section of the dam shows that it was constructed on natural river sediments, which were washed out due to piping. Various remedial measures were taken in the period 1913 to 1919 to repair the damage and resolve continuing leakage problems. These measures ultimately resulted in a large amount of materials being placed on the upstream face to form an impervious blanket.

16. FERC obtained jurisdiction over the Elwha project in December 1978. DRB staff studies showed the dam to be unstable and to be a hazard to the downstream population, resulting in an order requiring the owner to file a plan for rehabilitating the project prior to licensing. In July 1980 a contract was awarded for posttensioning the dam and work was completed in October 1980. In June 1981, after a review of the as-built drawings and construction records, the owner was notified that the construction was not performed according to specifications and was, therefore, not acceptable. The owner was ordered to convene an independent board of consultants to rule on the adequacy of the posttensioning work. That Board ruled, in March 1985, that the posttensioning work did not meet specifications due to improperly placed anchorage grout, and recommended installation of additional anchors. The additional work is scheduled for completion by April 1986.

17. There are some unusual features of the design of the Elwha repairs. Driving forces on the dam were based on readings from two piezometers installed

through the dam into the upstream fill material. This resulted in a reduction in hydrostatic forces on the dam. Secondly, the dam was analyzed as a wedge in the valley. Due to the unusually large depth-to-span ratio, the gravity section is essentially a beam spanning the original river bed, with the abutments founded on competent rock. Lastly, weak planes within the body of the gravity section were stabilized by the installation of posttensioned anchors.

18. The remedial measures approved by FERC staff involved installation of 20 multistrand, high-strength steel tendons in the gravity section and spillway piers. A VSL system was used, with tendons spaced at 10 feet on center, and stressed to approximately 300 kips. In addition, a third piezometer was required in order to provide for more reliable monitoring of the hydrostatic pressures in the upstream fill material. See Figures 1 and 2.

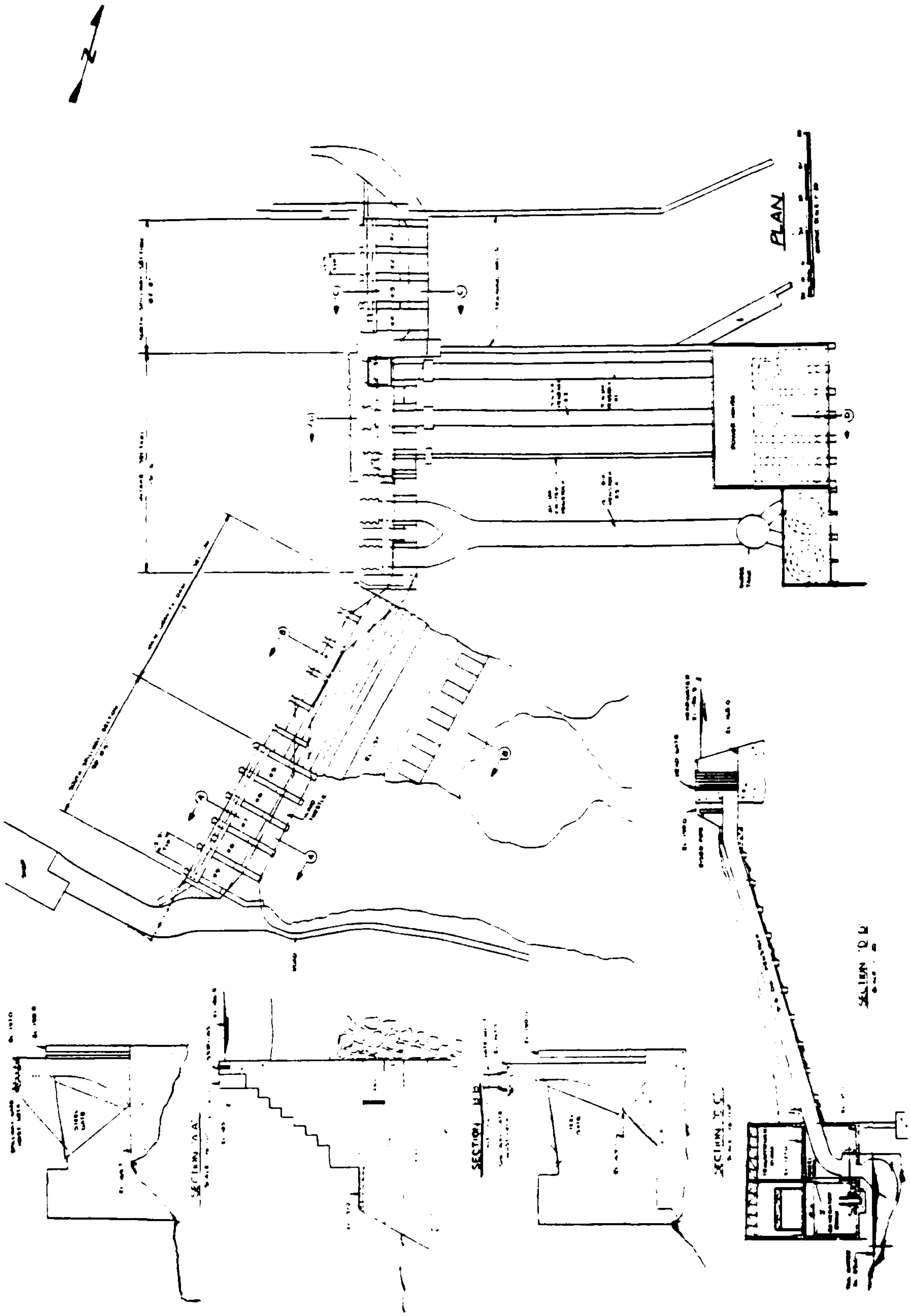
Claytor Dam

19. The Claytor Dam, located on the New River, near Radford, Virginia, was constructed in 1939. It is comprised of an overflow gated spillway, a powerhouse, and nonoverflow abutment sections. The total length of the dam is 1142 feet and has a maximum height of 120 feet. See Figure 3.

20. Claytor Dam was first identified as requiring a safety evaluation through the Part 12 inspection program and was required, by DRB Staff, to file a plan for rehabilitation as a condition for relicensing in 1980. The dam was found to be unstable for PMF loading, and a remedial action plan was filed in June 1983. A two-phase plan was approved, which involved raising the spillway gages to increase spillway capability and stabilizing the nonoverflow sections of the structure.

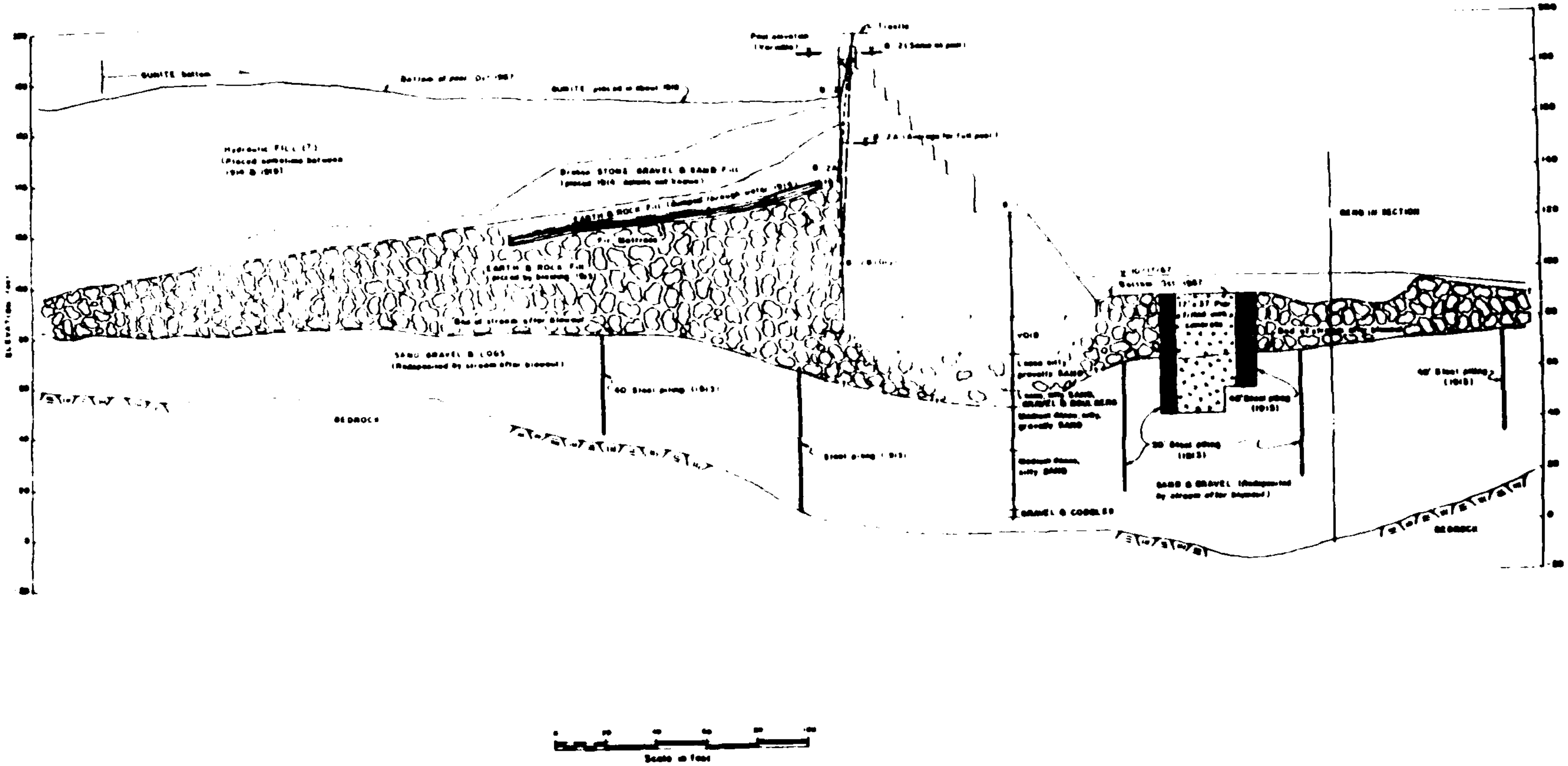
21. Phase one rehabilitation, raising of the spillway gates, was approved for construction in September 1983, and work was completed in July 1984. The gates were raised 27 feet to increase spillway capacity and to thereby lower the PMF levels (by approximately 6 feet) in the reservoir as well as loads on all structures. This lowered the hydrostatic loads on the spillway so that it would be stable under the PMF (Figure 4).

22. Phase two rehabilitation included posttensioning of a weak plane in the right nonoverflow abutment and the installation of reinforced concrete thrust blocks at both abutments. In addition, rock downstream of both abutments was



ELWHA HYDROELECTRIC PROJECT

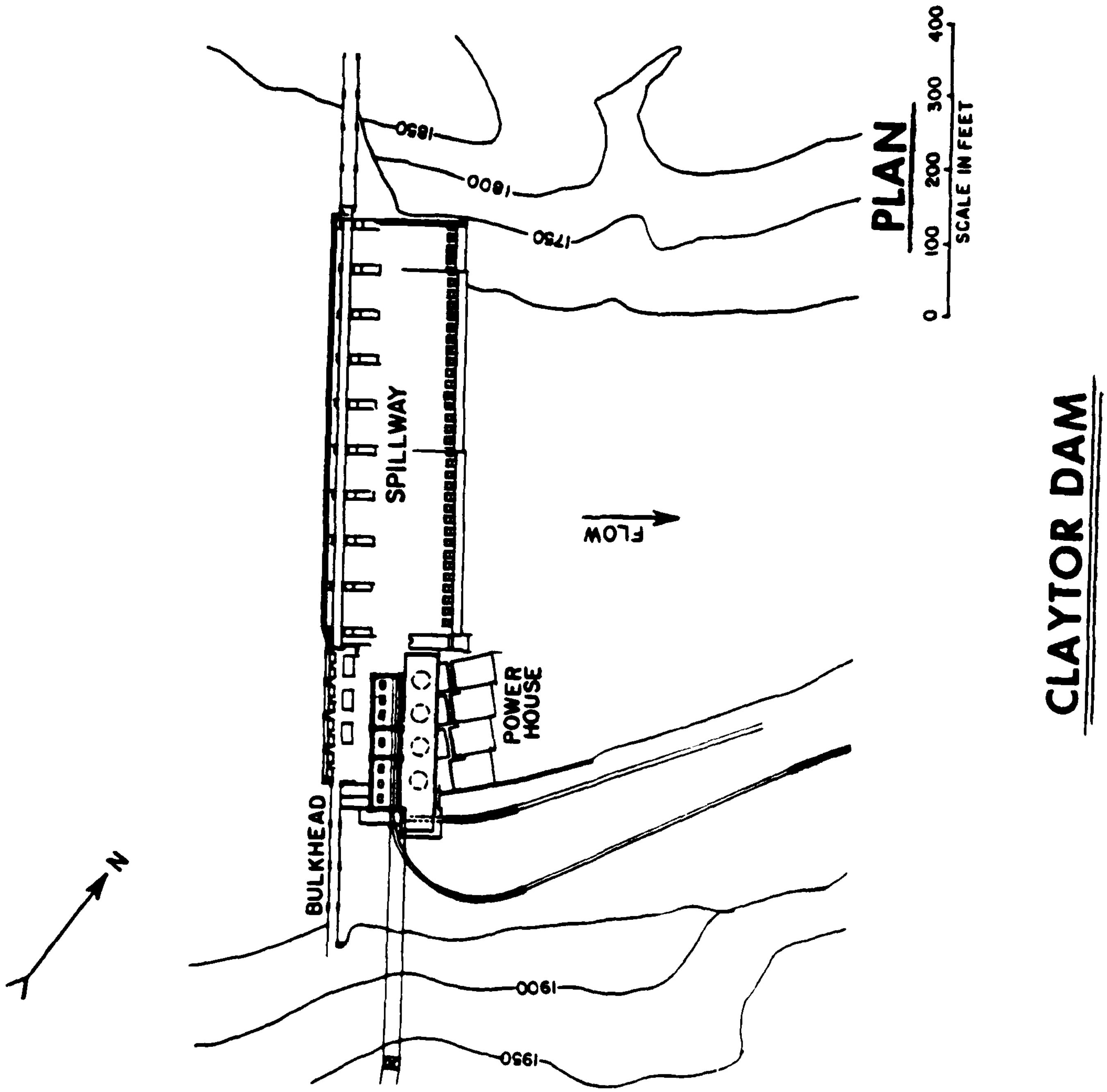
Figure 1



ELWHA HYDROELECTRIC PROJECT

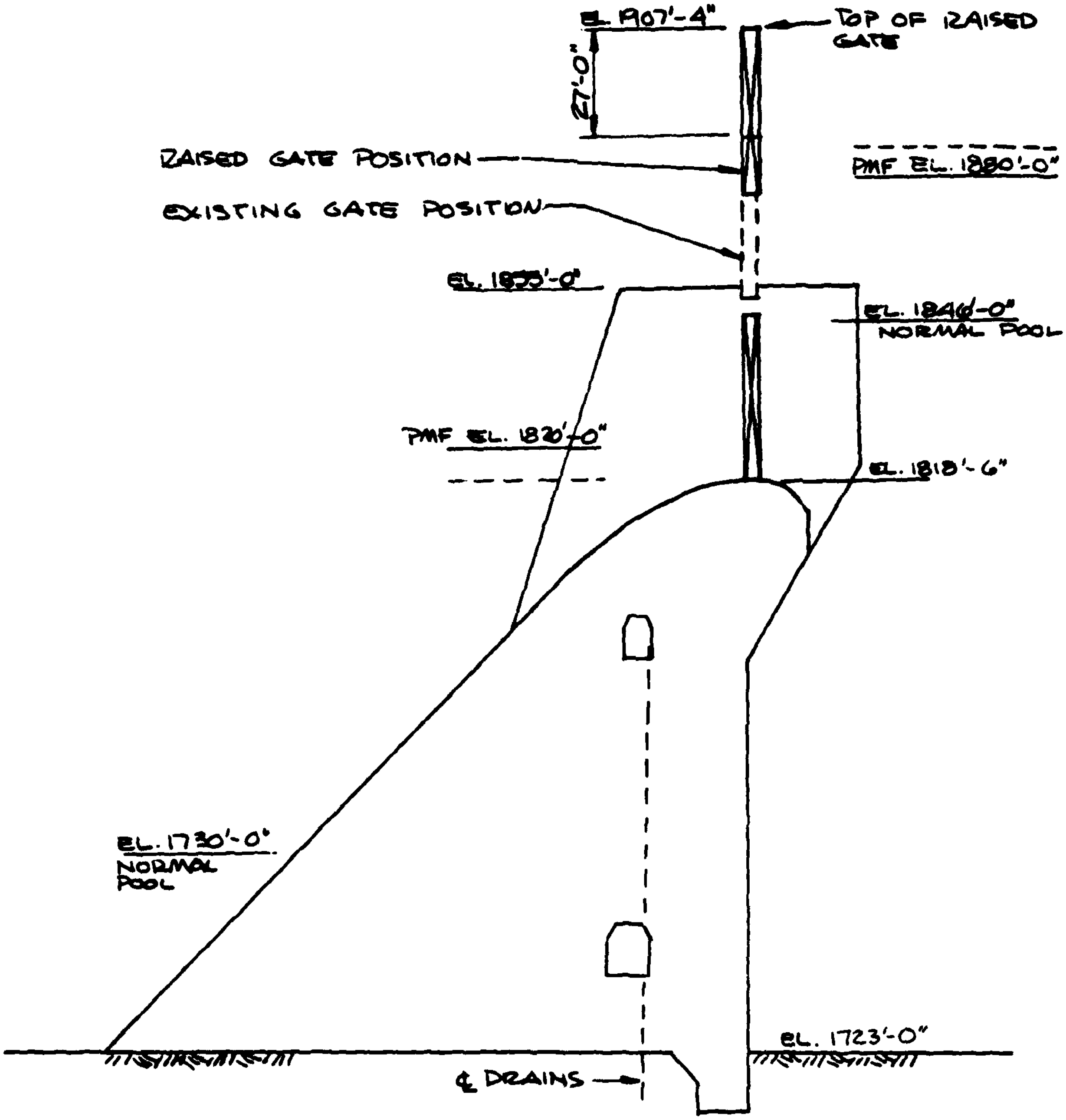
CROSS - SECTION OF DAM

Figure 2



CLAYTOR DAM
PLAN
SCALE IN FEET
0
100
200
300
400
SPILLWAY
POWER HOUSE
BULKHEAD
FLOW
N
1850
1800
1750
1900
1950

Figure 3



CLAYTOR DAM
GATE RAISING

Figure 4

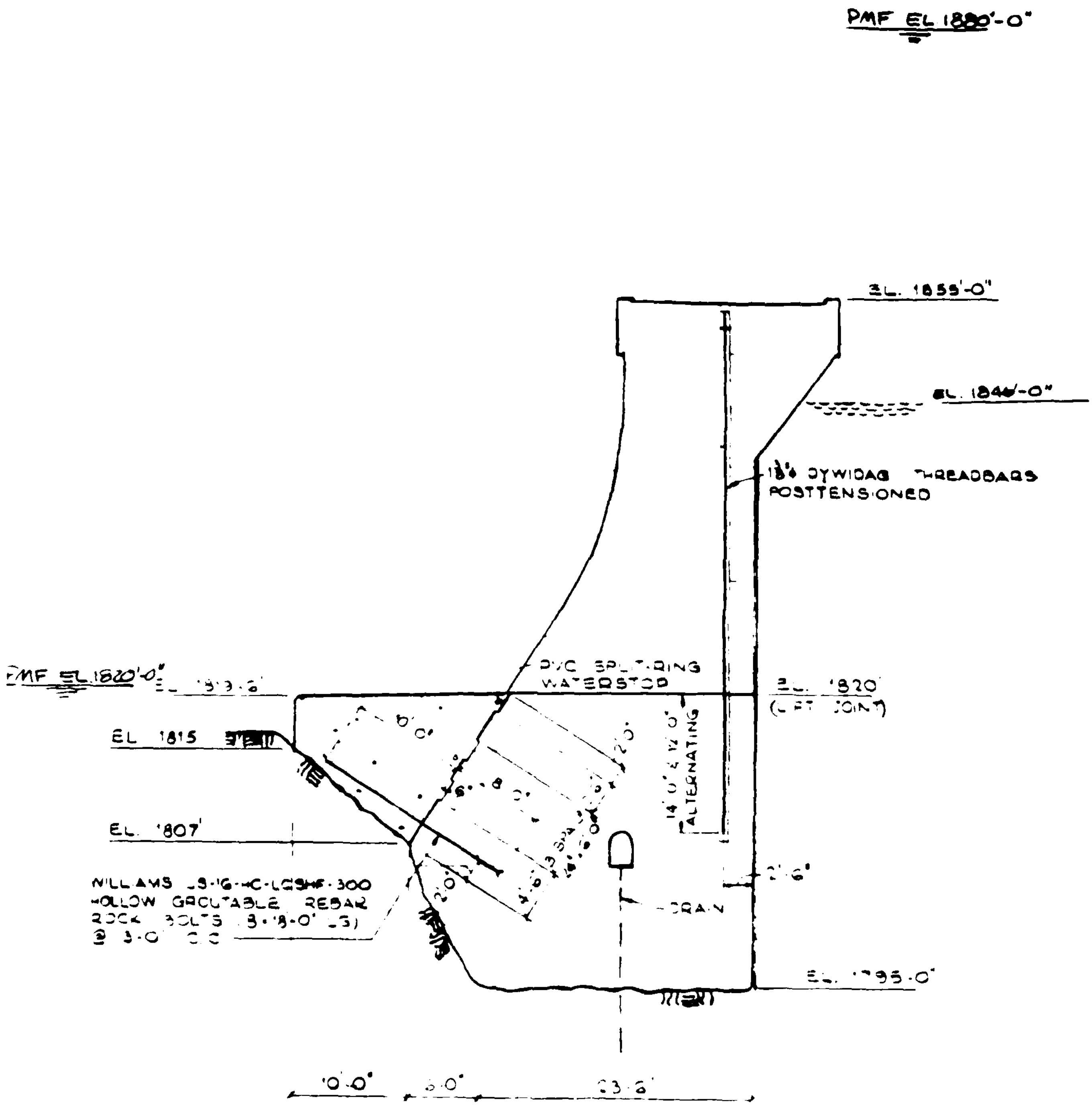
grouted and capped with concrete to prevent unraveling during overtopping of the structure. This work was completed in March 1985 (Figure 5).

23. Both phases involved unusual solutions to the dam safety problem. The raising of the gates, in phase one, allowed for the PMF to be passed through the spillway more efficiently, but did not increase capacity sufficiently to prevent overtopping. This required that all sections be analyzed for loadings induced by overtopping, and raised many questions concerning the performance of and existing drainage system under hydrostatic loading far exceeding anything previously experienced by the system. Phase two stabilization of the nonoverflow portions was complicated by geologic conditions which precluded posttensioning to the foundation rock. A thrust block of reinforced concrete was approved which was designed to resist the tensile forces induced by the resultant of all forces falling outside the original base of the dam. Posttensioning was used to stabilize a plane in the right abutment section which was cracked and leaking, but the anchors did not extend into the foundation (Figures 6 and 7).

Barker Dam

24. Barker Dam is a 175-foot-high, 270-foot-long concrete gravity dam located on the Middle Boulder Creek, near Boulder, Colorado. It was constructed in 1910 and developed excessive leakage to the extent that in 1933 the dam and foundation were grouted. Leakage again developed and in 1947 additional repairs were made, consisting of the installation of a closed drainage system, placement of a concrete membrane on the upstream face, and deep foundation grouting (Figure 8).

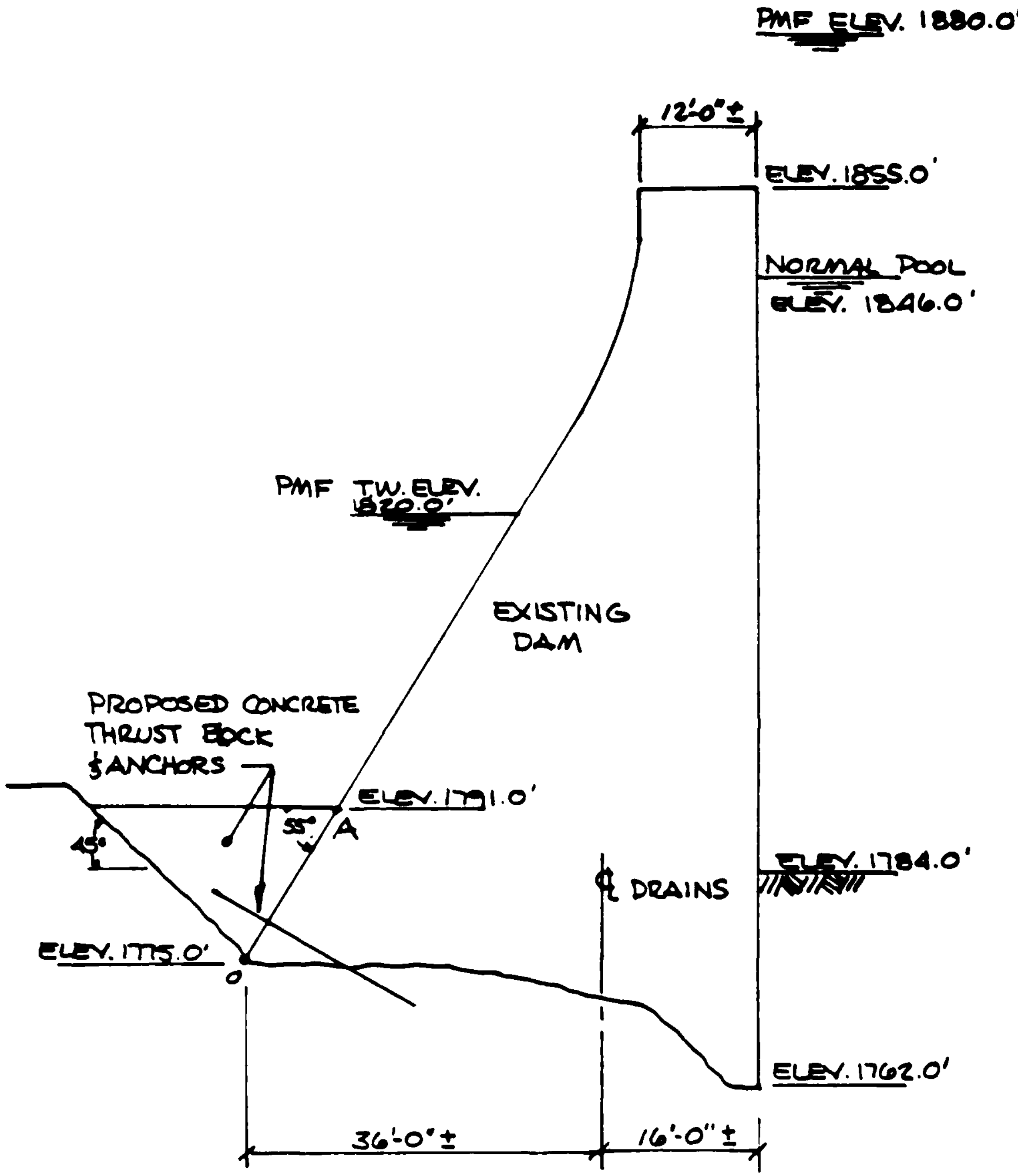
25. Concerns for the stability of Barker Dam were again raised in the DRB studies as part of the relicensing in 1981 and the licensee was ordered to undertake a study to determine the actual uplift distribution under the dam. Staff studies showed the dam to be unstable under PMF and normal loading conditions, using conventional uplift assumptions. The licensee conducted a comprehensive geotechnical investigation including piezometer and Gloetzl cell installation. This study showed that uplift pressures existed under portions of the dam which approached the theoretical cracked-base assumptions used in the staff studies; i.e., a significant percentage of the available head existed over portions of the base. Other portions of the base were shown to



CLAYTOR DAM

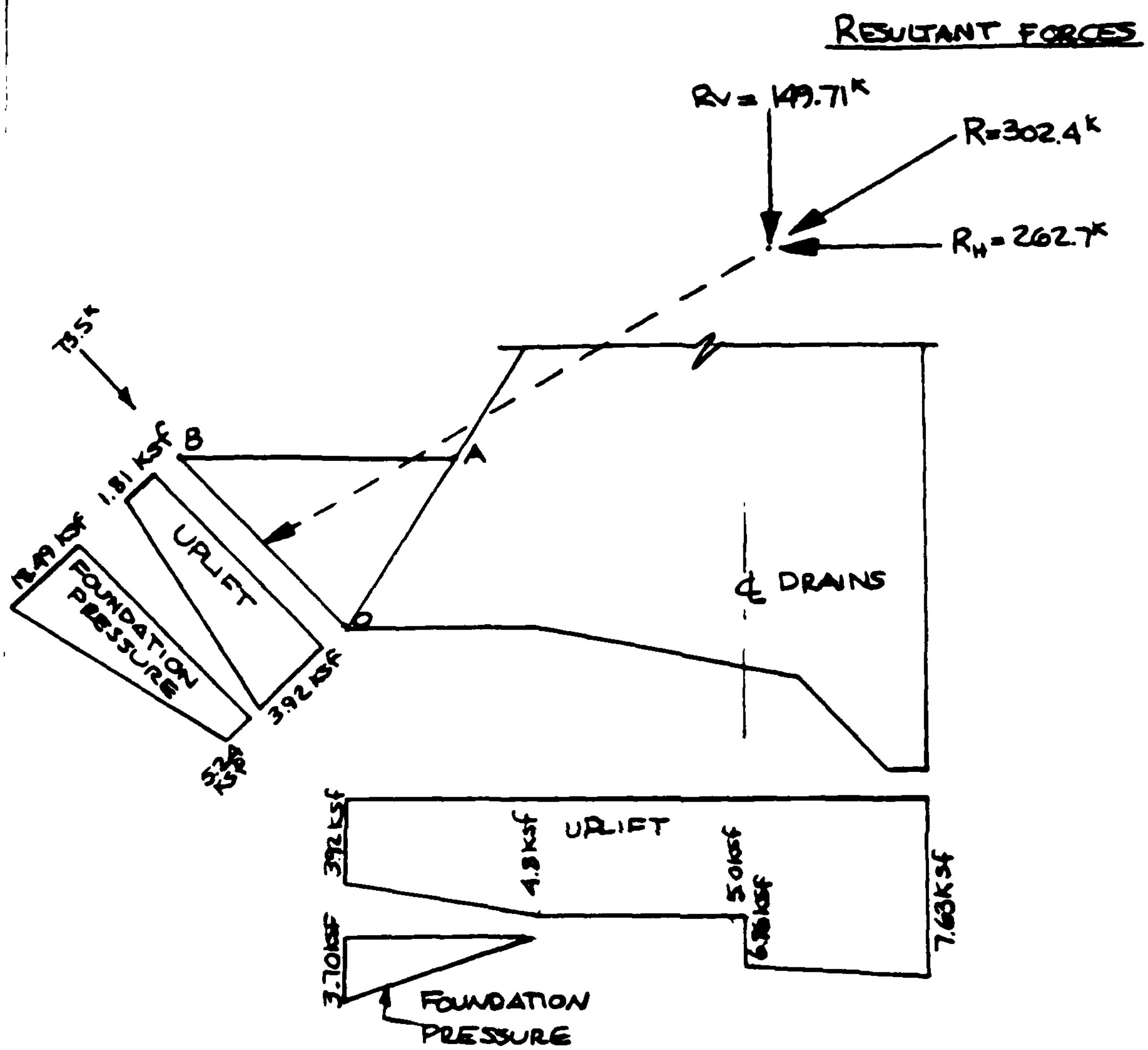
RIGHT ABUTMENT

Figure 5



CLAYTOR DAM

Figure 6



FOUNDATION REACTIONS

CLAYTOR DAM

Figure 7

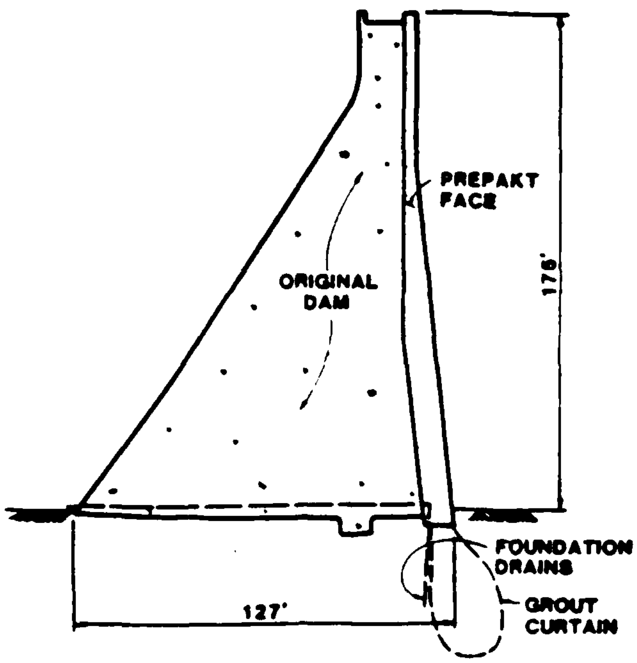


Fig. 1 - 1946 Restoration

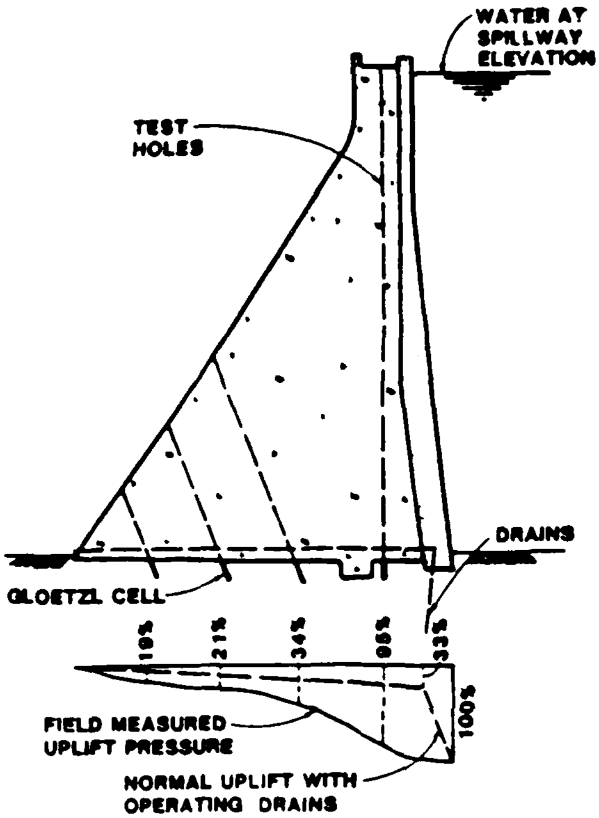


Fig. 2 - 1982 Uplift Pressures from Field Investigation

BARKER DAM

Figure 8

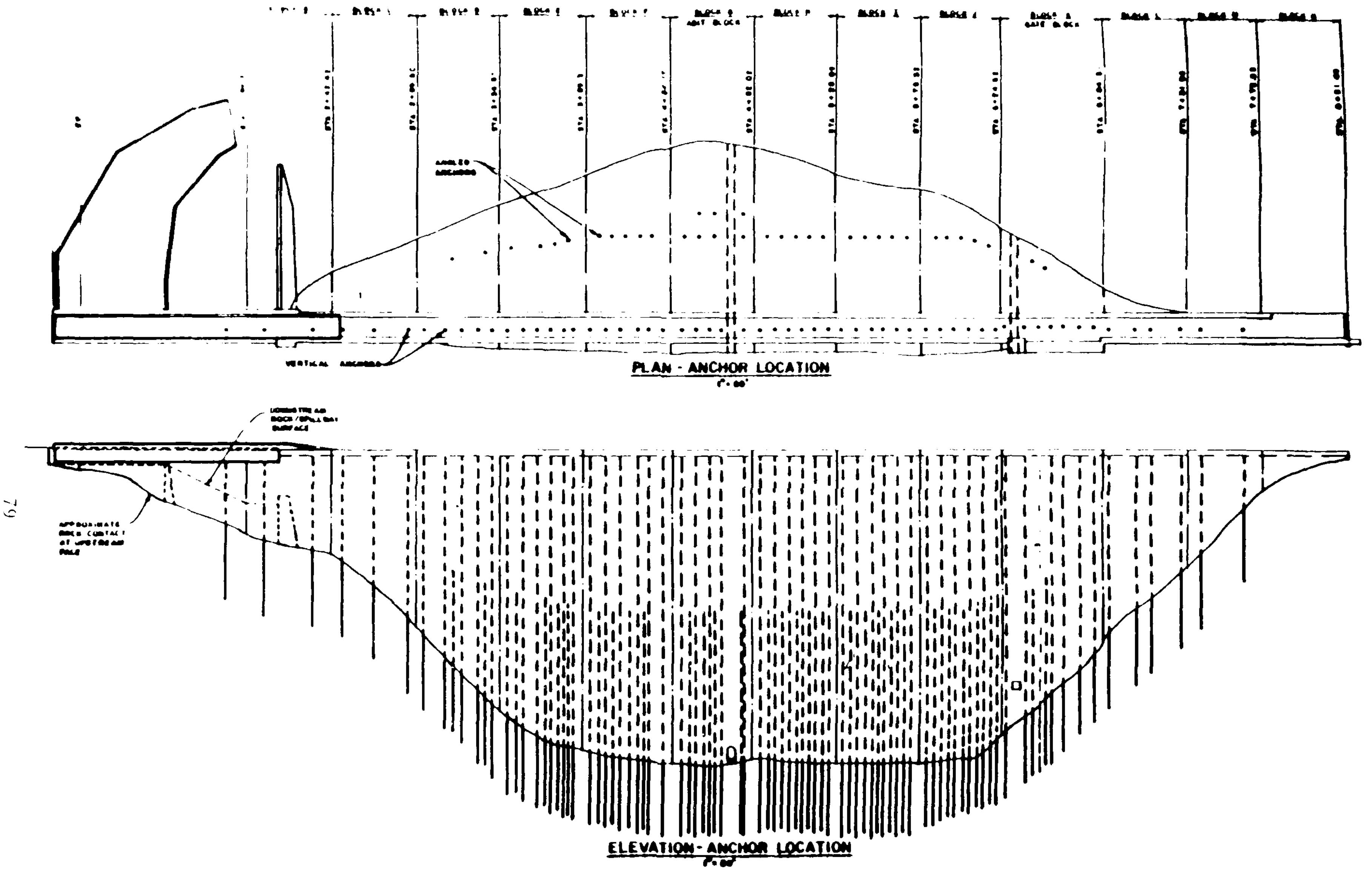
have pressures in the range of 15 to 33 percent of the available head when the reservoir level was raised slightly.

26. The conditions outlined above prompted the Commission to order the reservoir lowered by 19 feet to increase stability until repairs could be implemented. Repairs to Barker Dam are unusual due to the fact that the forces required to stabilize the structure would not allow placement of tendons at a spacing closer than 8 feet on center. This resulted in the installation of vertical anchors along the crest of the dam and additional rows of anchors placed at 30-degree angle through the downstream face (Figures 9 and 10). A finite element analysis was conducted to determine concrete and foundation stresses resulting from the posttensioning forces, and to determine interaction between anchors. The design required 94 multistrand, high-strength tendons stressed to design forces of up to 1450 kips. Each tendon consisted of 52, 1/2-inch-diameter 270-ksi strands. Work was completed in December 1984 at a cost of approximately $5,000,000.

Recommendations

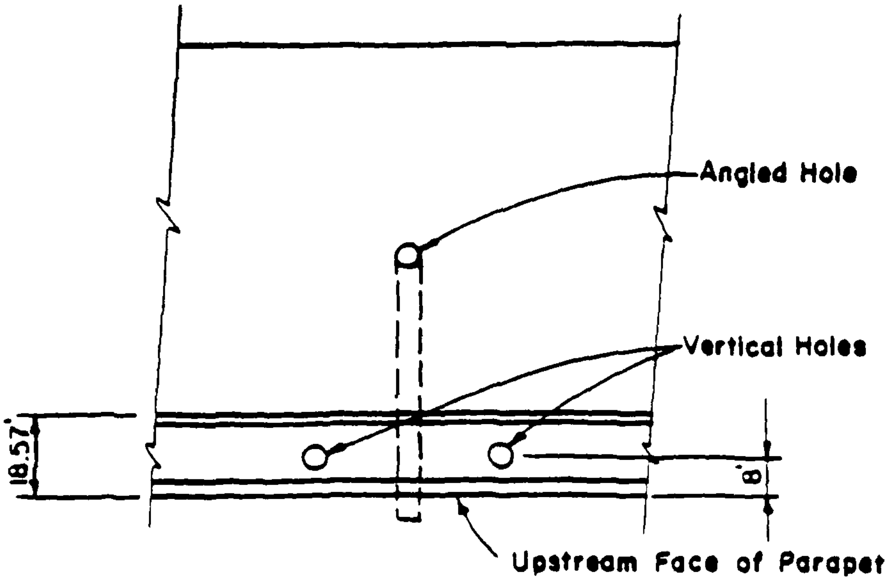
27. The safety evaluation of an existing gravity dam is a complex, site-specific problem with each project presenting unique conditions that sometimes require unusual solutions. This makes establishing generalized criteria and analysis procedures difficult. The FERC experience with gravity dams has, however, indicated several areas which we feel would benefit by additional research leading to the development of standard evaluation methods.

a. Uplift Pressures - Distribution and Analysis Methods. The magnitude and distribution of uplift pressures below an existing dam are probably the most critical elements in the stability analysis, and are unfortunately the most difficult to establish. Four specific areas require research and are described below as questions to be answered: (1) How are drain efficiencies affected by increasing the available head? Current practice is to use the same efficiency at all reservoir levels, but, as shown by the Barker Dam experience, this may not be conservative. Analysis procedures, recently published in the literature, require the assumption of the magnitude of the head at the entrance to a drainage system, without consideration of the system's ability to handle the inflow quantity. Research leading to guidelines

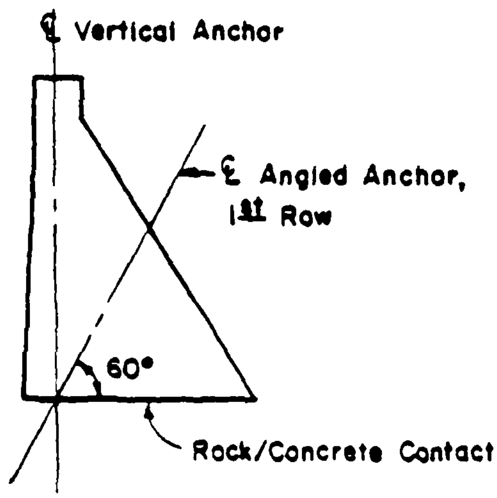


BARKER DAM

Figure 9



TYPICAL PLAN, HOLE LOCATIONS



TYPICAL SECTION

BARKER DAM

Figure 10

of extrapolating from drain efficiencies at normal conditions to efficiencies at much greater inflow pressures and quantities would be beneficial.

(2) How does a cracked interface at higher than normal pool elevations affect drain efficiency? The performance of the drainage system when the base crack extends beyond the location of the drains is extremely critical to the analysis (Figures 11 and 12). This condition requires the application of full head-water pressure for an undrained base, but current criteria do not give guidance for a cracked-base with drainage.

(3) How should uplift be considered in an analysis, as a load or separately as a pressure to determine the initiation of interface cracking? Corps procedures use uplift as a load on the structure in the same way as driving forces, but USBR procedures separate uplift from the analysis. For design purposes the two procedures are identical because tension at the heel is avoided by changing the structural geometry so that the resultant of all forces is within the kern of the base, and low drain efficiencies are usually used. For existing structures, however, the two methods can give dramatically different conclusions when a high drain efficiency is used. See appendix for an example.

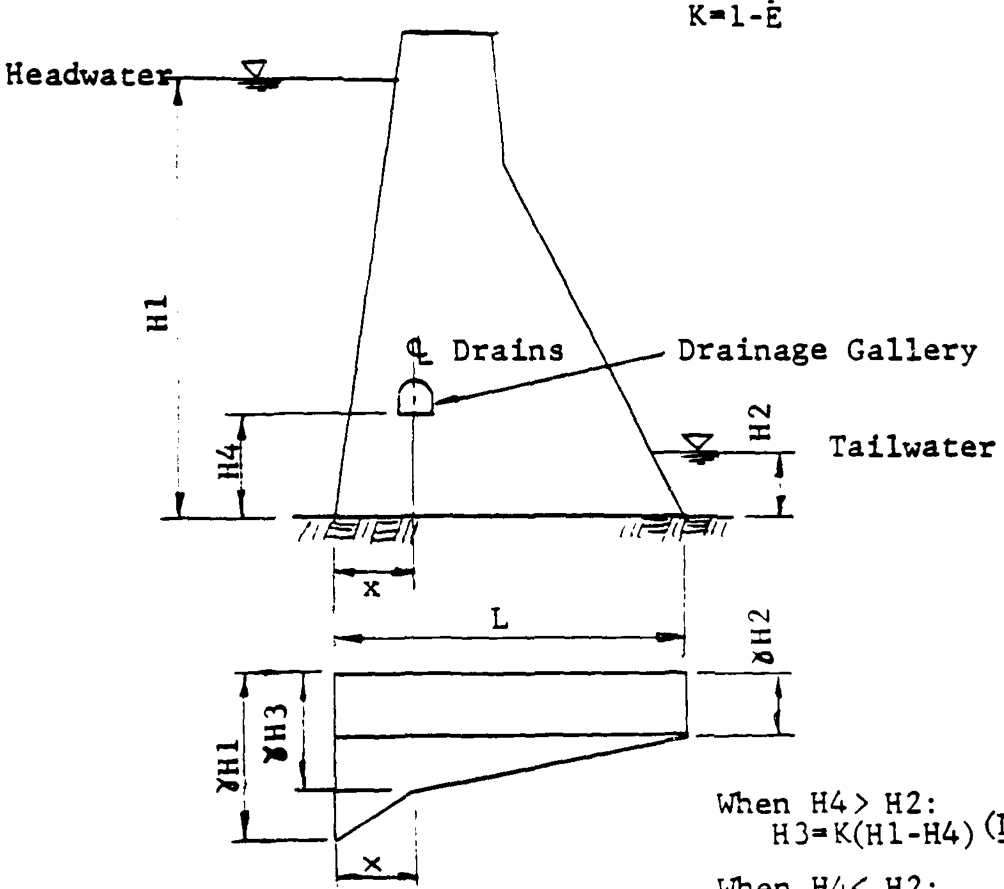
(4) How should uplift be considered in a finite element analysis? Several procedures are currently used, from applying the effective uplift forces as point loads on the interface to using pore pressures below an assumed phreatic line.

b. Interface Strength Parameters. The strength of the rock-concrete interface, in both shear and tension, is equally difficult to establish and almost as important to the analysis as uplift distribution. Research which establishes guidelines in the following areas would be beneficial:

(1) Recommendations concerning the testing techniques and extent of geologic investigation required to establish strength parameters for an existing dam.

(2) Establishment of guidelines for selection of strength parameters based on expected rates of loading or/and probability of a particular loading condition occurring. For instance, what parameters should be

where;
E=Drain effectiveness
expressed as a decimal
K=1-E



When H4> H2:

$$H3=K(H1-H4)\frac{(L-X)}{L} + H4$$

When H4< H2:

$$H3=K(H1-H2)\frac{(L-X)}{L} + H2$$

UPLIFT DISTRIBUTION

UNCRACKED BASE WITH DRAINAGE

Figure 11

when $H4 < H2$

$$H3 = K(H1-H2)\frac{(L-T1-x)}{L-T1} + H2$$

when $H4 > H2$

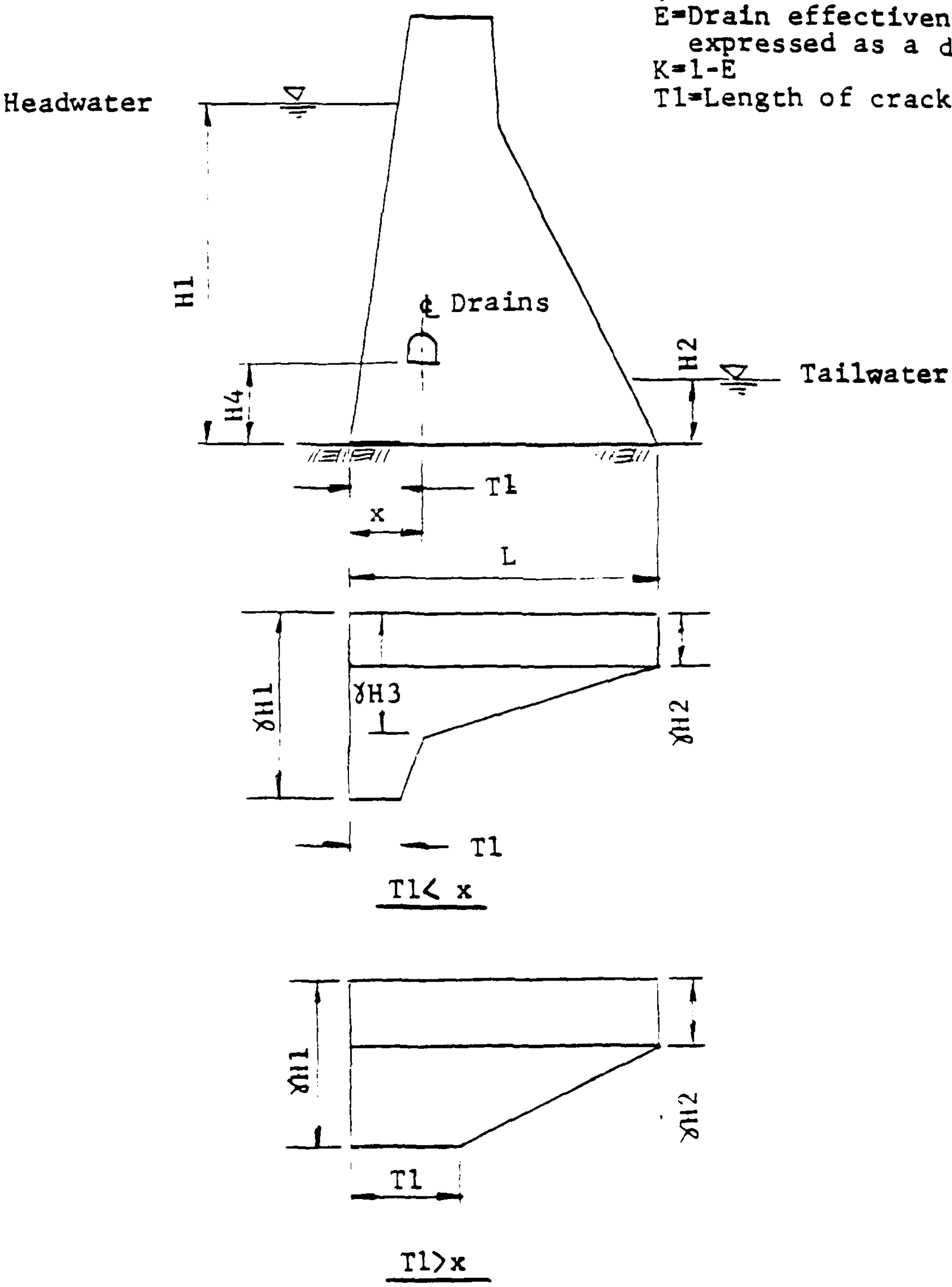
$$H3 = K(H1-H4)\frac{(L-T1-x)}{L-T1} + H4$$

where;

E=Drain effectiveness expressed as a decimal

K=1-E

T1=Length of crack



UPLIFT DISTRIBUTION

CRACKED BASE WITH DRAINAGE Figure 12

used in an earthquake analysis when the expected loading is of higher magnitude and much shorter duration than normal conditions?

c. Establish Guidelines of Safety Evaluation of Existing Dams. The establishment of a consistent set of guidelines and criteria to be used by all federal agencies in the safety evaluation of existing gravity dams would be beneficial, as would recommendations concerning when and to what degree upgrading of unsafe dams is required.

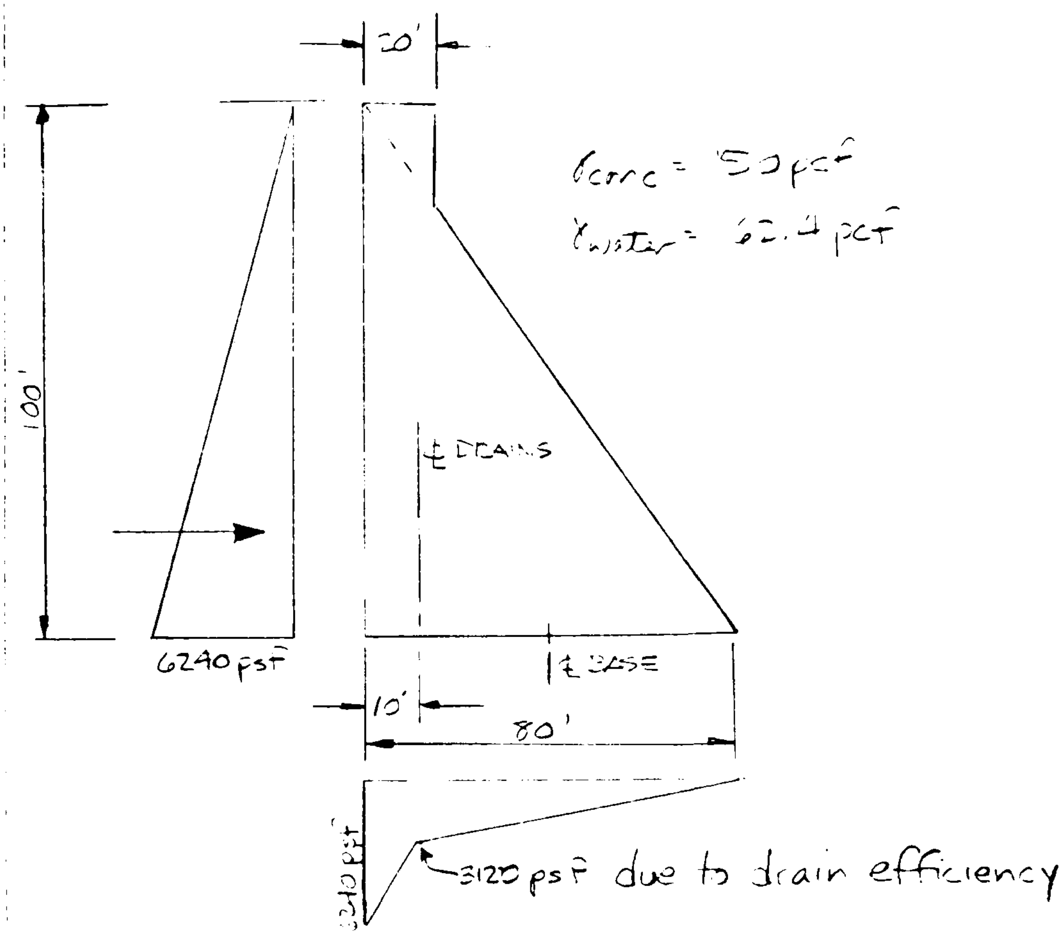
(1) Criteria for evaluating dam safety is generally that established by the major dam builders in government, such as the Corps, USBR, and TVA. Therefore, inconsistencies among the agencies concerning the procedures and criteria for dam safety evaluations (e.g., uplift analyses as previously mentioned) have an impact on the entire industry. The primary benefit would be that a consistent level of public safety would be assured.

(2) The development of criteria and procedures for determining when and to what degree a structure should be upgraded would be beneficial because the economic impact on private and municipal dam owners would be reduced. The current system of each agency having independent standards of evaluation sometimes requires owners to satisfy conflicting levels of dam safety concern.

APPENDIX

Gravity Dam Example

Uplift Assumptions



Item	Weight (Kips)	Moment Arm (Ft)	Moment Abt. ℄ (k-ft)
Driving Force $\frac{100}{1000}(62.4)100(\frac{1}{2})$	312.0 →	33.33	10,400.0
Dam = $100(80)\frac{1}{2}\frac{150}{1000}$	600.0 ↓	13.33	8000.0
ΣV =	600.0k ↓	ΣM =	2,400.0
Σ→ =	312.0k →		
Uplift $\frac{3120}{1000}(70)\frac{1}{2}$	109.2 ↑	6.67	728.0
$3120(10)\frac{1}{2}$ =	15.6 ↑	36.67	572.0
3120(10) =	31.2 ↑	35.0	1092.0
ΣU =	150.0k ↑	ΣM_u =	2392.0 k-ft

Corps Method

$\Sigma M = 2400.0 + 2392.0 = 4792.0^{k\text{-}ft}$

$\Sigma V = 600 - 156 = 444^{k} \downarrow$

$\Sigma M / \Sigma V = 10.79\ ft. \qquad L/6 = 13.33\ ft.$

Base Stresses

$heel = \frac{444}{80} - \frac{4792.0(6)}{80^2} = 5.55 - 4.49 = \underline{1.06}\ ksf$ compression

$toe = 5.55 + 4.49 = \underline{10.04}\ ksf$ compression

$\boxed{\text{Entire Base in Compression}}$

Bureau Method

$\Sigma M = 2400.0^{k\text{-}ft}$

$\Sigma W = 600^{k}$

$e = 4.0\ ft.$ to right of ℄

$heel\ pressure = \frac{600}{80}\left(1 - \frac{6(4)}{80}\right) = \underline{5.25\ ksf}$

$uplift\ pressure = 6.24\ ksf > 5.25\ ksf$

$\boxed{\text{therefore base is Cracked}}$

$e' = \frac{2400}{600 - 6.25(80)} = \underline{24.0'}$

$T_1 = 3(40 - 24) = \underline{48}$ ft of base is cracked

$BS = \frac{2(600 - 6.25(80))}{48} + 6.24 = \underline{10.41}\ ksf$ toe pressure

Corp Method yields no cracking, and Bureau method yields a 32 ft (40% of base) crack.

IF uplift is not reduced by drainage

$$U = \frac{6240}{1000}(80)\frac{1}{2} = 249.6^k$$

$$M_u = 249.6\left(\frac{80}{6}\right) = 3328.0^{kft}$$

Corps' Method

$$\Sigma M = 2400 + 3328 = 5728.0^{kft}$$

$$\Sigma V = 600 - 249.6 = 350.4^k \downarrow$$

$$\Sigma M/\Sigma V = 16.35' \qquad L/6 = 13.33 ft.$$

Base stresses

$$heel = \frac{350.4}{80} - \frac{5728.0(6)}{80^2} = 4.38 - 5.37 = -.99 \text{ ksf tension}$$

$$toe = 4.38 + 5.37 = 9.75 \text{ ksf compression}$$

Portion of base not in compression would now be subjected to full uplift and an iteration process would begin to determine crack length.

Bureau Method

By inspection base is cracked since 5.35 < 6.24 in previous example.

Results will be the same as before, i.e.

$e = 24.0 ft.$
$I_1 = 48.0 ft.$

$BS = 10.41$ ksf

The two methods yield the same results when uplift is not reduced by drainage.

COMPUTER CODES AVAILABLE TO ASSIST
IN STABILITY ANALYSIS

AD-P003689

N. Radhakrishnan

US Army Engineer Waterways Experiment Station

1. The Three-Dimensional Stability Analysis/Design Program (3DSAD) has been developed under the Computer-Aided Structural Engineering (CASE) project for the analysis and design of hydraulic structures with respect to overturning, sliding, and maximum bearing. Mr. Fred Tracy, Automation Technology Center, WES, is the principal author of this program. The program has two modes of operation: (1) general structural capability and (2) specific structure types.

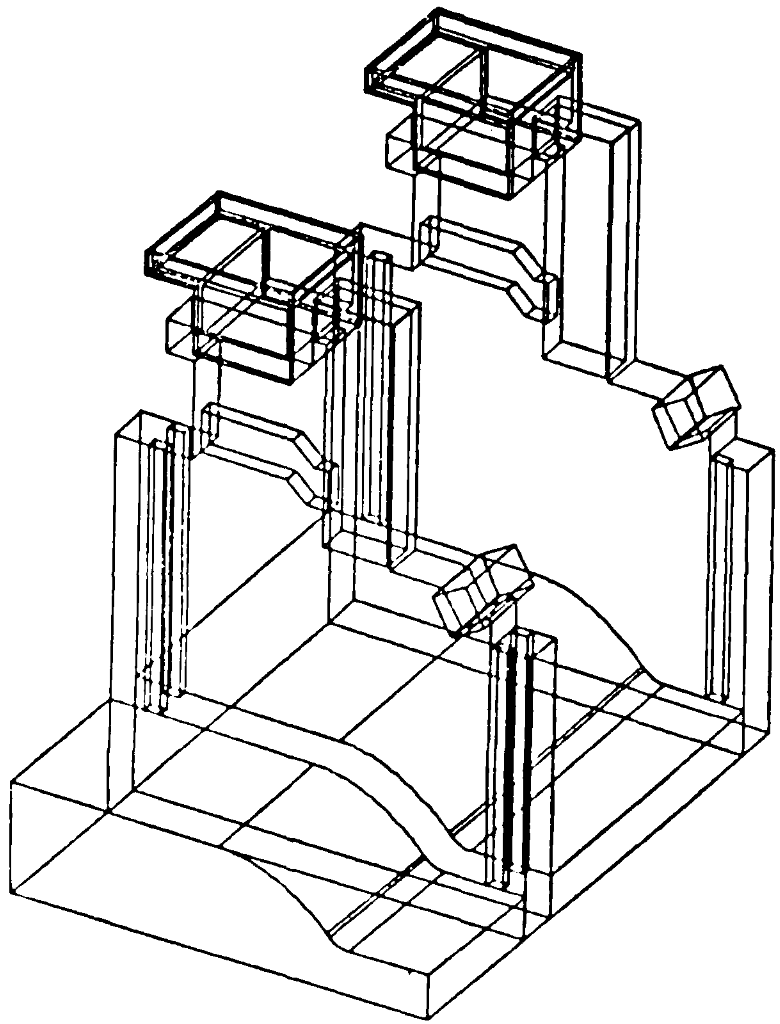
2. The general modules allow the user to create and interactively plot the geometry and loads of the structure, obtain volumes, weights, centroids, forces, and moments for these, and then do an analysis to determine if the base pressures, area of base in compression, sliding, and maximum bearing are at acceptable values. A Free-Body Module also exists to "clip" the geometry and loads to determine a new problem and make the subsequent analysis.

3. The specific structure modules (currently dams and gravity locks) start with certain predetermined shapes (the dams module, for instance, has an overflow cross section, a nonoverflow cross section, and a pier section) and allows the user to give specific values for the parameters for these pieces. The general geometry, loads, and analysis data are then automatically generated for the user. Also specific load cases are preprogrammed in the specific structures modules. For instance, the dams module has the six standard load cases of construction, normal operating condition, induced surcharge, flood, and the two earthquake conditions. A Design Memorandum (DM) plate capability is also available for dams.

4. This program has been used by several Corps of Engineers District offices on a variety of projects. Examples are Lock & Dams 2 and 26 by the St. Louis District, Richard B. Russell Dam by the Mobile District, and Chief Joseph Dam by the Seattle District.

5. Some of the slides used to describe and discuss the capabilities of 3DSAD follow:

OPERATING HOUSES ON TOP OF DAM PIERS, LOCK & DAM #2



ISOMETRIC VIEW

COMPUTATION

1. Volumes, weights, and centroids
2. Forces and moments

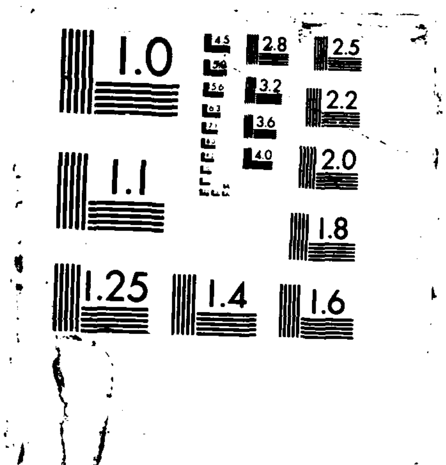
MASS PROPERTIES TABLE

NO.	NAME	VOLUME	WEIGHT	XCG	YCG	ZCG
1	S1	46185.720	6927.858	35.693	40.356	4.000
2	S2	73931.429	11089.714	32.059	8.892	38.000
3	S3	46185.720	6927.858	35.693	40.356	72.000
4	S4	36965.714	5544.857	32.059	8.892	91.000
5	S5	580.952	87.143	64.999	56.707	5.790
6	S6	761.058	114.159	64.999	56.707	72.005
7	S7	-504.945	-75.742	25.882	69.193	4.000
8	S8	-3085.933	-462.890	43.155	79.292	4.000
9	S9	-504.945	-75.742	25.882	69.193	72.000
10	S10	-3085.933	-462.890	43.155	79.292	72.000
11	S11	307.250	46.088	25.198	97.924	-6.750
12	S12	2110.219	316.533	25.622	94.902	4.000
13	S13	307.250	46.088	25.198	97.924	14.750
14	S14	307.250	46.088	25.198	97.924	61.250
[illegible]	S15	2110.219	316.533	25.622	94.902	72.000
[illegible]	S16	307.250	46.088	25.198	97.924	82.750
[illegible]	S17	-279.000	-41.850	7.500	43.250	7.000
[illegible]	S18	-279.000	-41.850	7.500	43.250	69.000
[illegible]	S19	-279.000	-41.850	7.500	43.250	75.000
[illegible]	S20	-252.000	-37.800	70.500	29.000	7.000
[illegible]	S21	-252.000	-37.800	70.500	29.000	69.000
[illegible]	[illegible]	-252.000	-37.800	70.500	29.000	75.000
[illegible]		301285.275	30192.791	33.418	23.317	47.671

AD-A185 644 PROCEEDINGS OF REMR (REPAIR EVALUATION MAINTENANCE AND REHABILITATION RES...(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS STRUC... W F MCCLEESE

2/3

UNCLASSIFIED JAN 87 F/G 11/2 NL



CROCOPY RESOLUTION TEST CHART

FORCE AND MOMENT TABLE

NAME	FX MX	FY MY	FZ MZ
WT	0.	0.	-138874.23
	-3402418.66	19681255.75	0.
V210	0.	0.	-7216.07
	-176793.61	665196.94	0.
V250	0.	0.	-1782.37
	-43668.08	415606.52	0.
WH10	72771.13	0.	0.
	0.	59138668.50	-1782892.58
WH50	-3382.53	0.	0.
	0.	-2556065.88	82872.02
PUN1	0.	0.	1050.94
	25747.98	-104275.90	0.
NWN1	0.	0.	234.36
	5741.71	-20093.94	0.
GAT1	0.	0.	-218.60
	-5355.70	27360.63	0.
MAC1	0.	0.	-16.00
	-392.00	1937.12	0.
BRI1	0.	0.	-1076.00
	-26362.00	118360.00	0.
TOTAL	69388.59	0.	-147897.97
	-3623500.34	77367950.00	-1700020.55

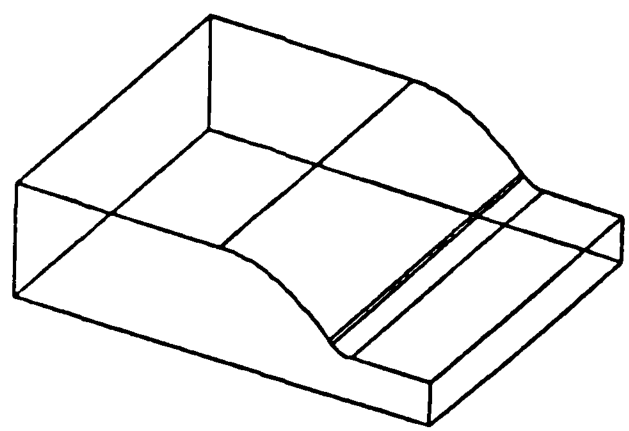
GENERAL GEOMETRY

1. Blocks
2. Bricks
3. Faces

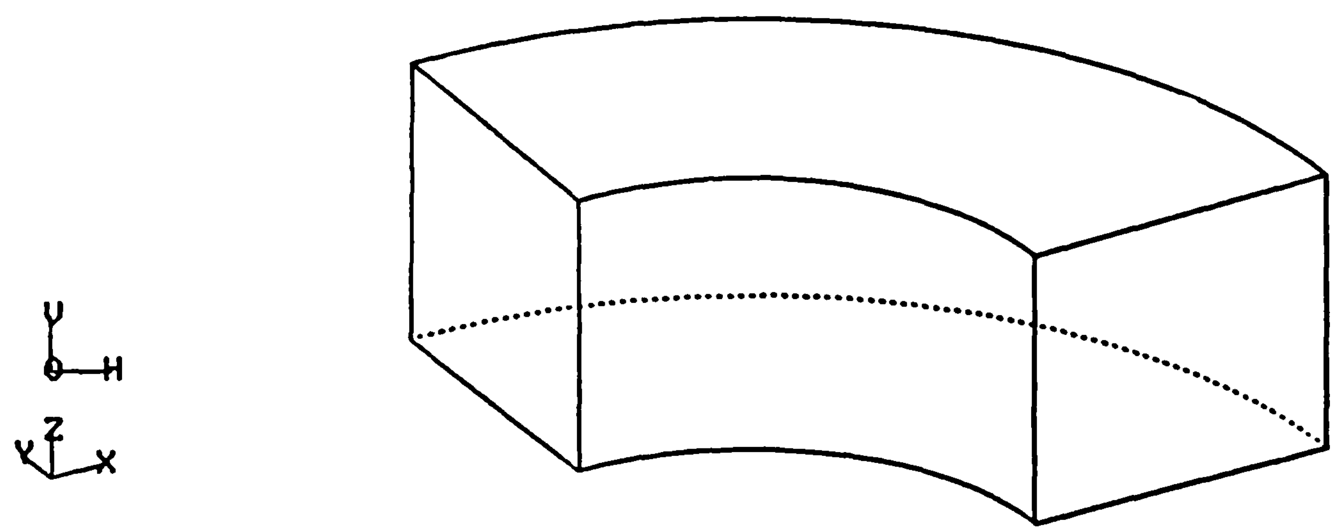
BLOCK

1. Define cross-section
2. Grow in 3rd dimension
    a. Constant
    b. Linear
    c. Quadratic
    d. Axisymmetric (new)

EXAMPLE BLOCK WITH CURVED EDGES ISOMETRIC VIEW

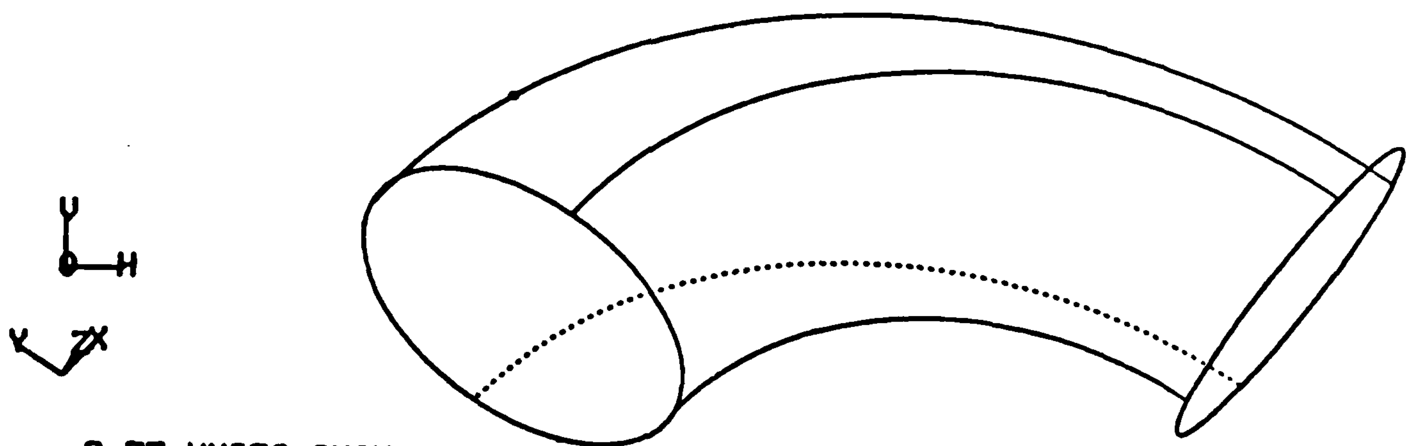


EXAMPLE BLOCK WHERE CROSS-SECTION IS SWEPT AXISYMMETRICALLY



SF = 3.14 UNITS/INCH

EXAMPLE BLOCK WHERE CROSS-SECTION WITH CURVED EDGES IS SWEPT AXISYMMETRICALLY

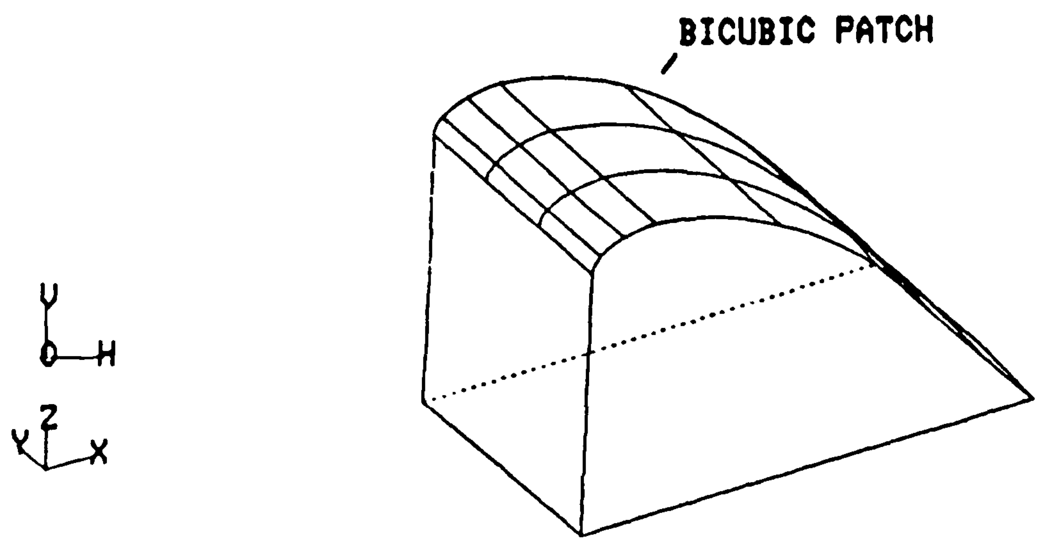


SF = 2.77 UNITS/INCH

FACES

1. *Planar face*
2. *Bicubic patch (new)*

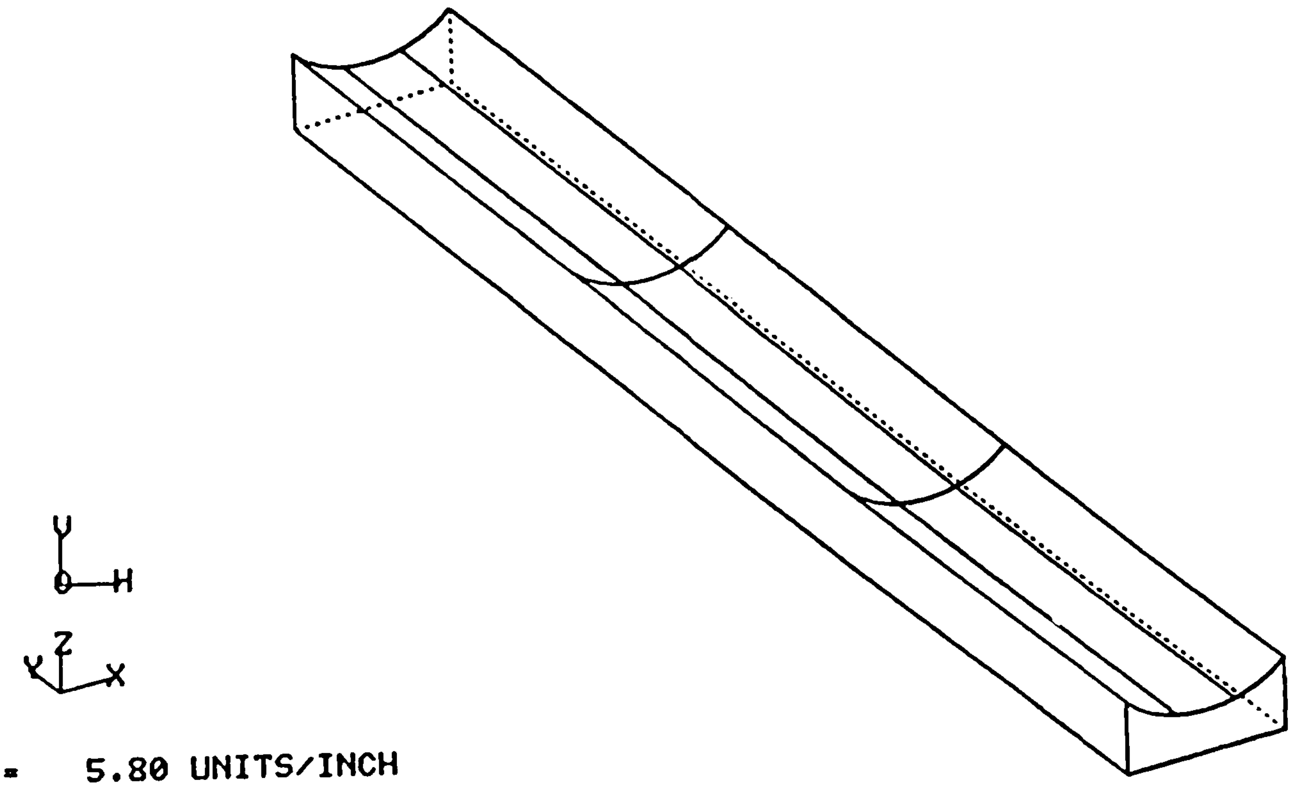
FACE DEFINITION OF PIECE USING PLANAR AND BICUBIC PATCHES



DATA GENERATION

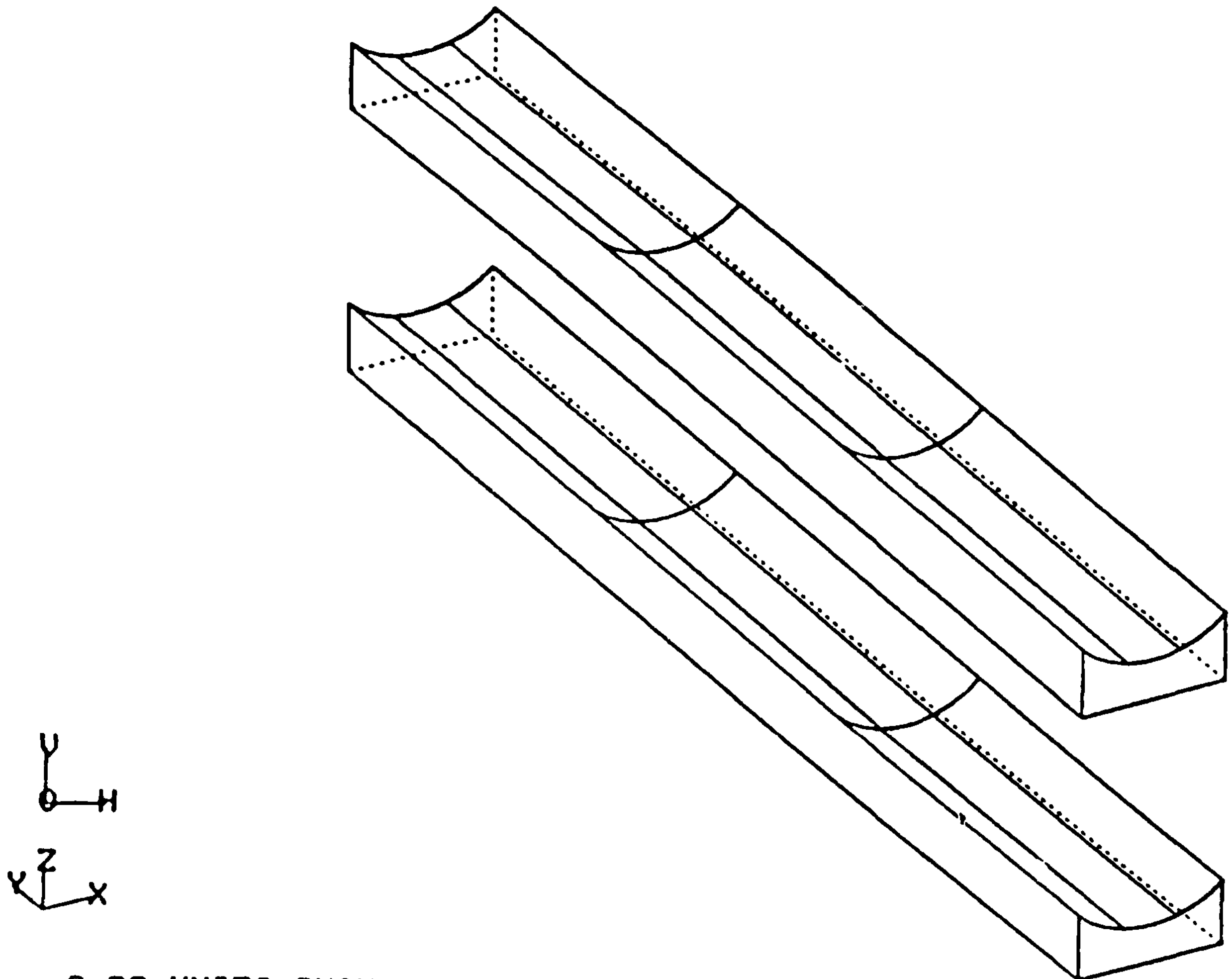
1. Translate
2. Rotate (new)
3. Copy (new)
4. Reflect (new)

ORIGINAL BLOCK



SF = 5.80 UNITS/INCH

DUPLICATE BLOCK AFTER BEING COPIED TRANSLATED



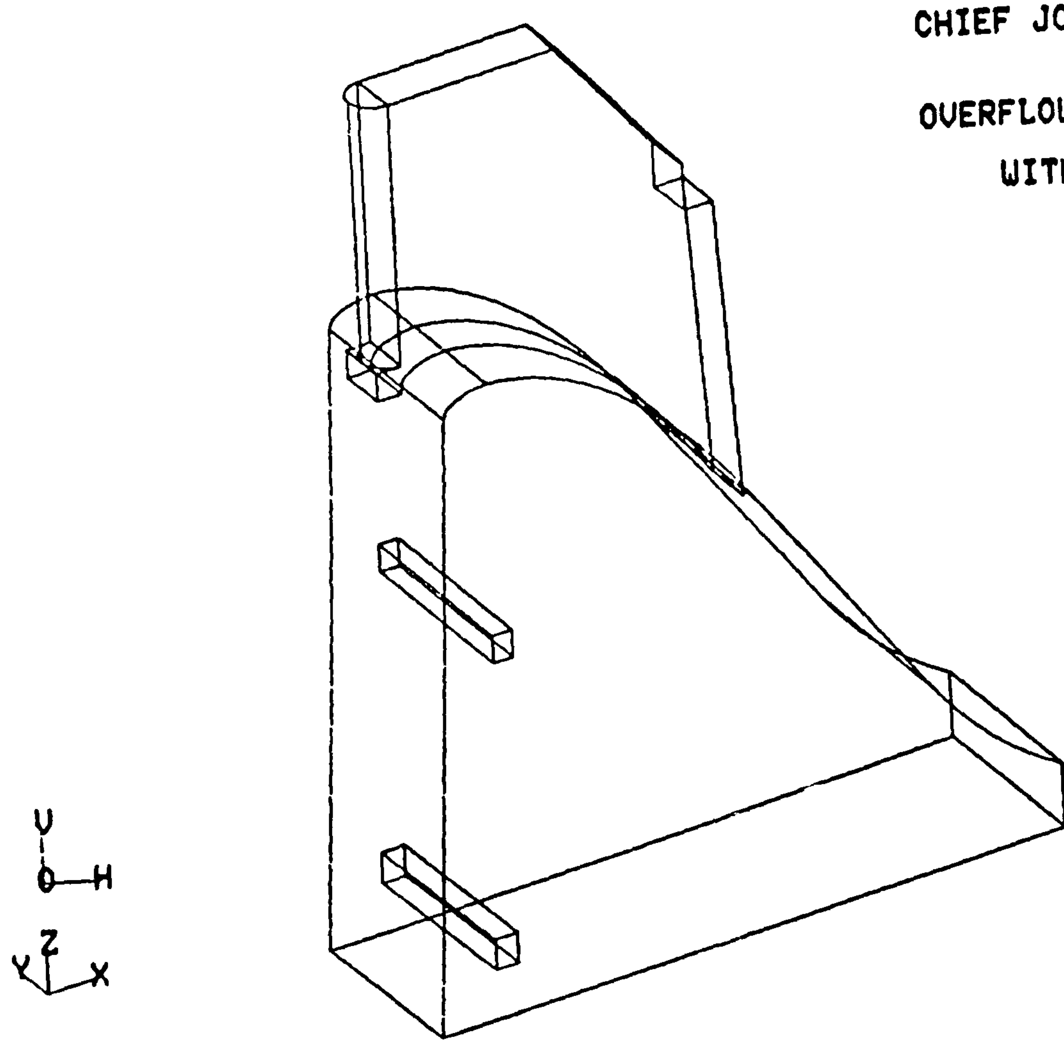
SF = 6.38 UNITS/INCH

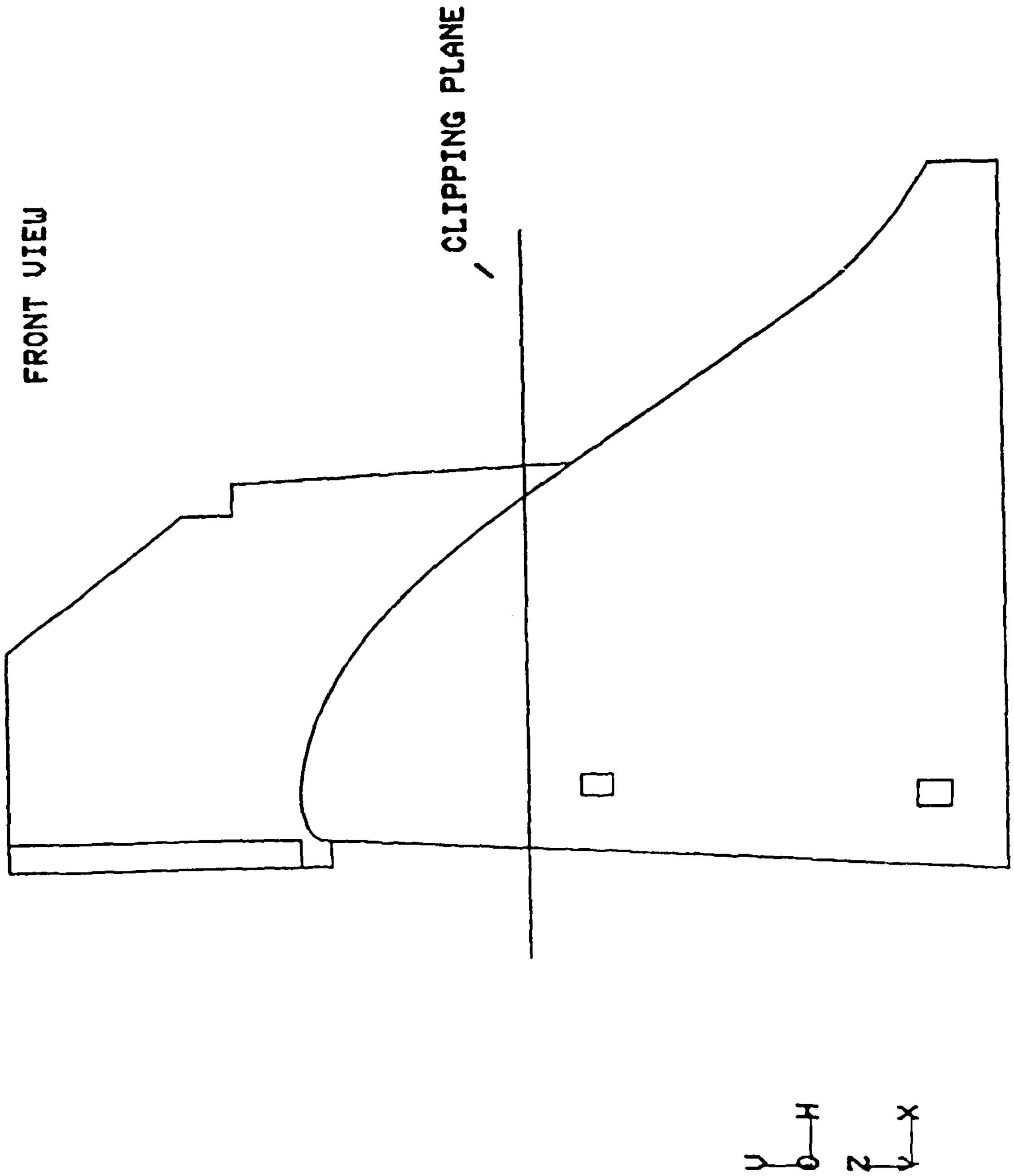
CLIPPING (NEW)

1. Arbitrary plane
2. Bicubic patches used for curved pieces

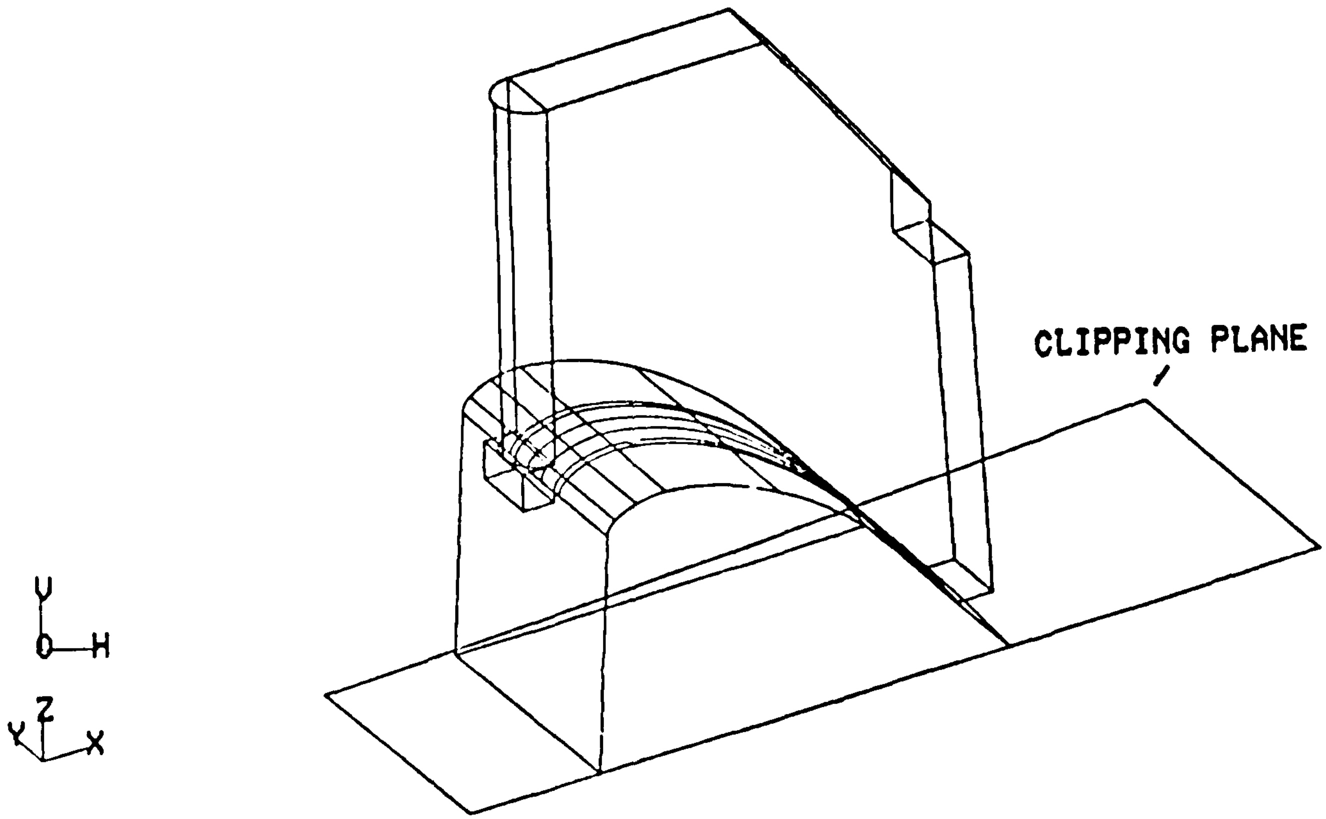
CHIEF JOSEPH DAM

OVERFLOW X-SECTION
WITH PIER




FRONT VIEW
CLIPPING PLANE

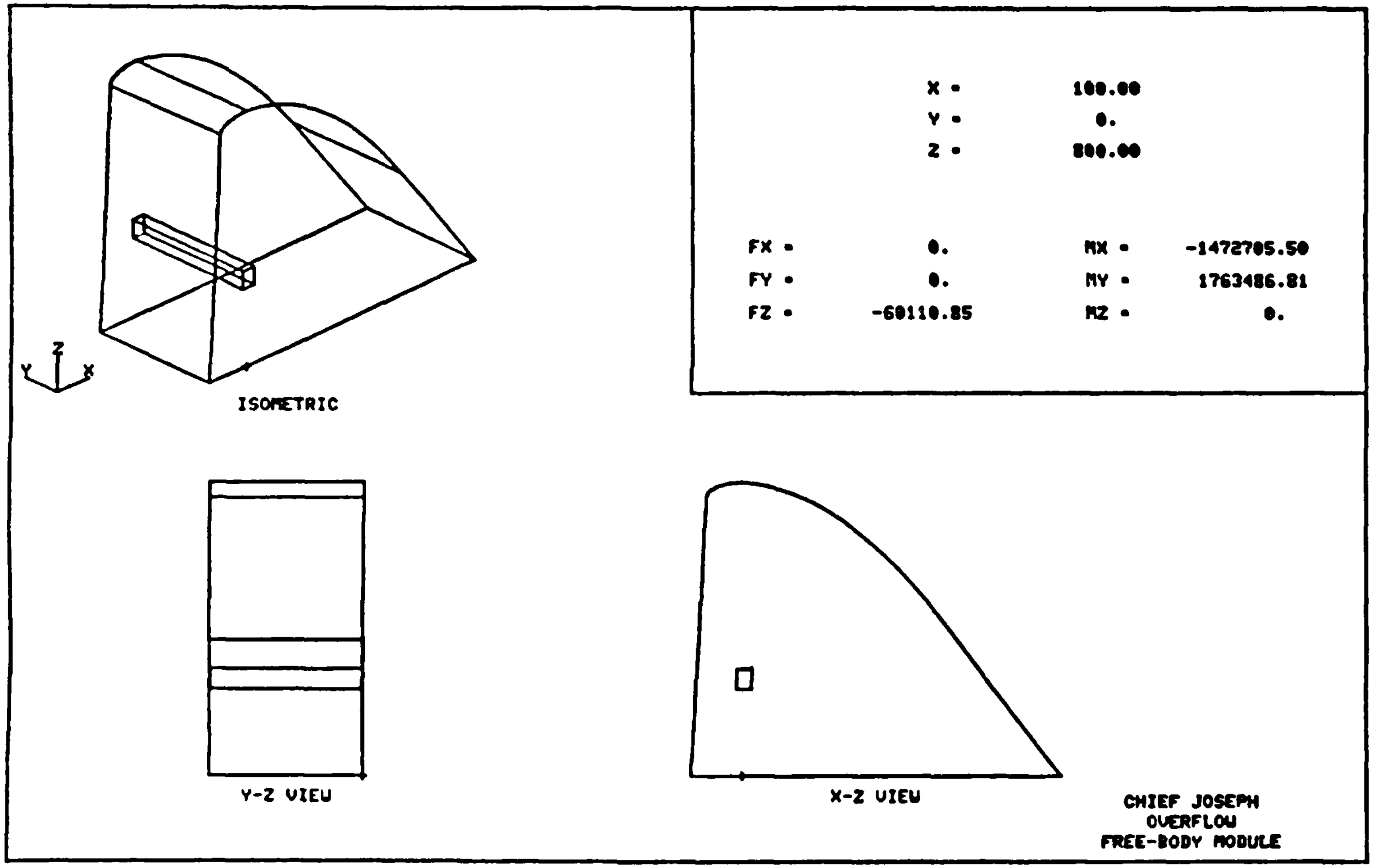
ISOMETRIC VIEW
WITH PART BELOW CLIPPING. PLANE REMOVED



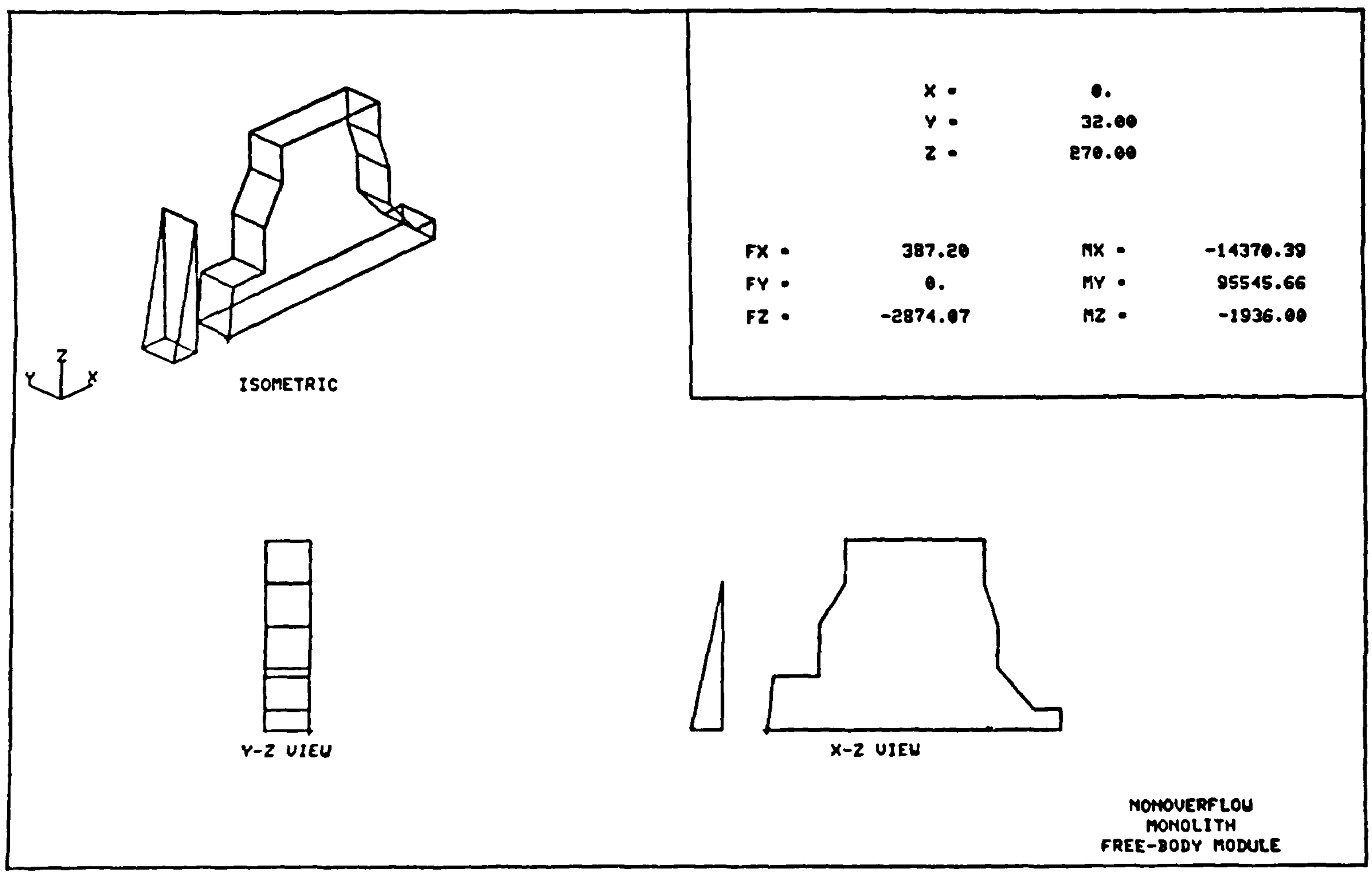
FREE BODY MODULE (NEW)

1. Uses clipping
2. Plots clipped structure
3. Computes forces and moments at a specified point

FREE-BODY MODULE RESULTS GEOMETRY LEFT FROM THE CLIP AND RESULTING FORCES AND MOMENTS



SECOND EXAMPLE OF FREE-BODY RESULTS WITH BOTH GEOMETRY AND LOADS



DAMS

1. Graphical input
2. Analysis
    a. Kern plot
    b. Base pressures
3. Design
4. DM Plate (new)

MENU FOR GRAPHICALLY INPUTTING AN OVERFLOW CROSS-SECTION FOR DAMS

OPTIONS

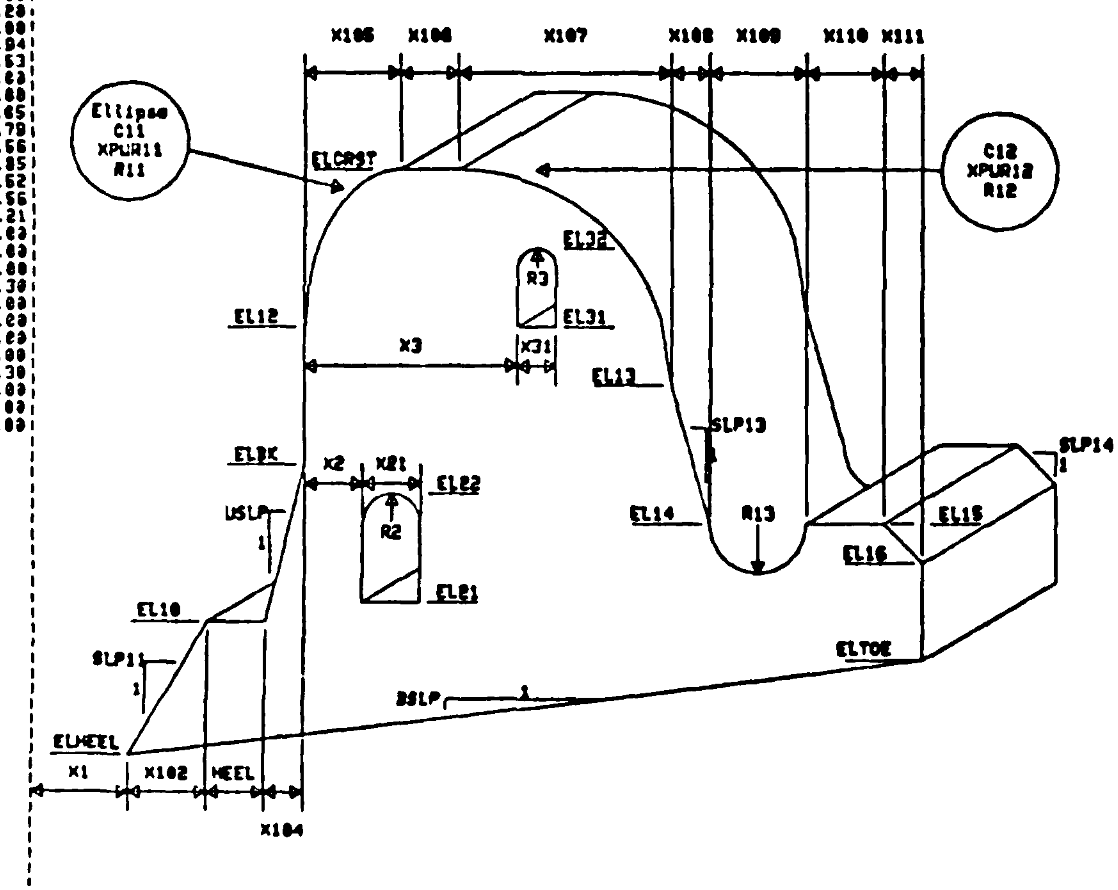
I - INPUT OR CHANGE DATA BY PLACING THE X-HAIRS OR CURSOR ON THE DESIRED LABEL IN THE PICTURE AND TYPING I.

ALL VARIABLES NOT X-ED OUT ARE 0.

P - PLOT PICTURE AND PRINT CURRENT DATA VALUES BY TYPING P.

G - SAVE DATA IN RESTART FILE AND GO BY TYPEING G.

ELHEEL	=	260.00
EL10	=	283.00
SLP11	=	0.10
HEEL	=	10.20
ELBK	=	283.00
EL12	=	288.94
X105	=	3.53
ELCRST	=	291.00
XPUR11	=	2.00
C11	=	6.65
X107	=	15.79
EL13	=	282.56
XPUR12	=	1.85
C12	=	19.62
EL14	=	262.56
X103	=	14.21
EL15	=	275.00
R13	=	20.00
ELTOE	=	260.00
X2	=	5.30
X21	=	2.00
EL21	=	275.00
EL22	=	279.00
R2	=	1.00
X3	=	13.30
X31	=	2.00
EL31	=	280.00
EL32	=	284.00



VARIABLE LIST

- X1
- XXXXXXX
- X102
- XXXXXXX
- XXXXXXX
- XXXXXXX
- X104
- XXXXXXX
- USLP
- XXXXXXX
- XXXXXXX
- XXXXXXX
- R11
- XXXXXXX
- XXXXXXX
- Ellipse
- X106
- XXXXXXX
- XXXXXXX
- R12
- XXXXXXX
- XXXXXXX
- X108
- XXXXXXX
- SLP13
- XXXXXXX
- XXXXXXX
- XXXXXXX
- X110
- X111
- EL16
- SLP14
- XXXXXXX
- BSLP
- XXXXXXX
- XXXXXXX
- XXXXXXX
- XXXXXXX
- XXXXXXX
- XXXXXXX
- XXXXXXX
- XXXXXXX
- XXXXXXX
- R3

MENU FOR GRAPHICALLY INPUTTING A PIER SECTION FOR DAMS

OPTIONS

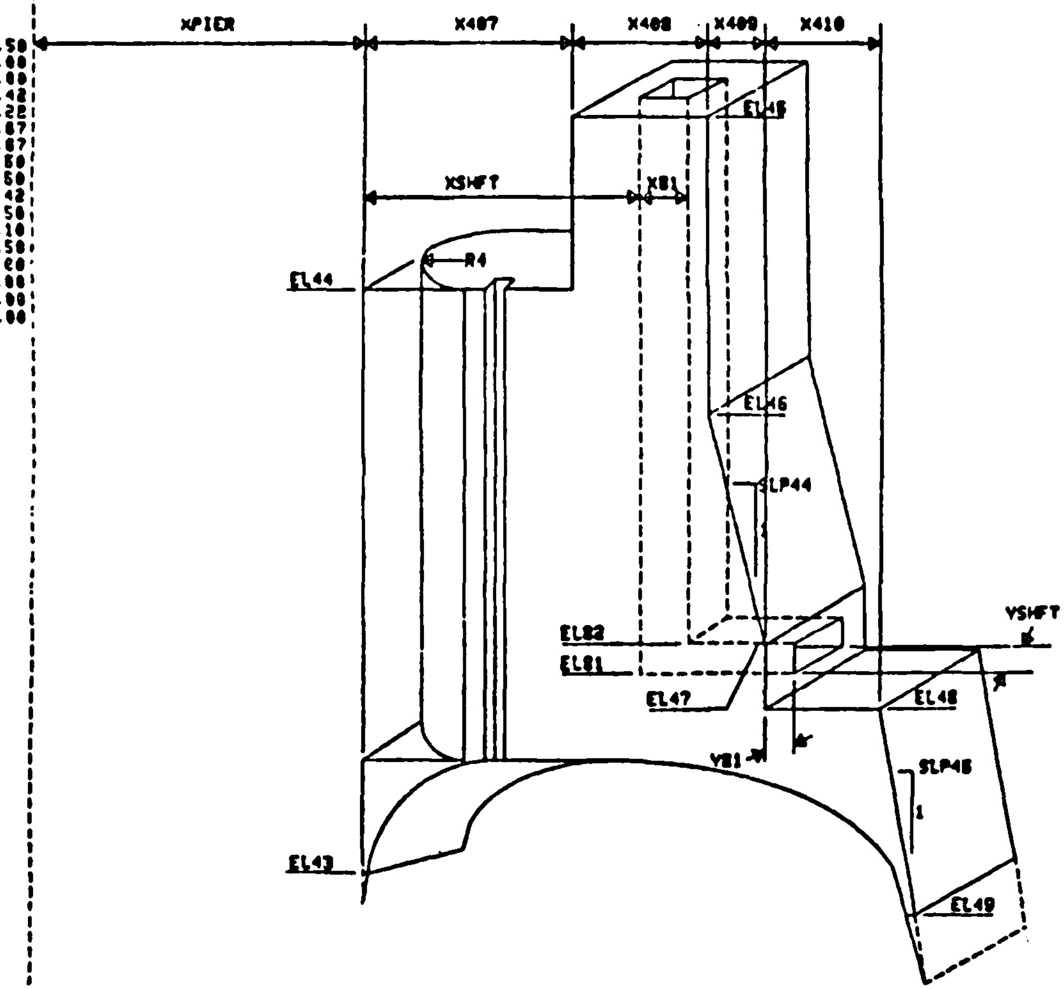
I - INPUT OR CHANGE DATA BY PLACING THE X-HAIRS OR CURSOR ON THE DESIRED LABEL IN THE PICTURE AND TYPING I.

ALL VARIABLES NOT X-ED OUT ARE 0.

P - PLOT PICTURE AND PRINT CURRENT DATA VALUES BY TYPING P.

Q - SAVE DATA IN RESTART FILE AND GO BY TYPING Q.

XPIER	=	1.50
EL43	=	283.00
R4	=	3.00
X407	=	18.42
EL44	=	311.22
X408	=	20.87
EL45	=	316.87
EL46	=	257.50
EL48	=	287.50
X410	=	4.42
EL49	=	277.50
SLP45	=	.10
XSHFT	=	28.50
YSHFT	=	4.00
XB1	=	3.00
EL81	=	259.00
EL82	=	301.00



VARIABLE LIST

EL47
X409
SLP44
YB1

MENU FOR GRAPHICALLY INPUTTING A NON OVERFLOW CROSS-SECTION FOR DAMS

OPTIONS

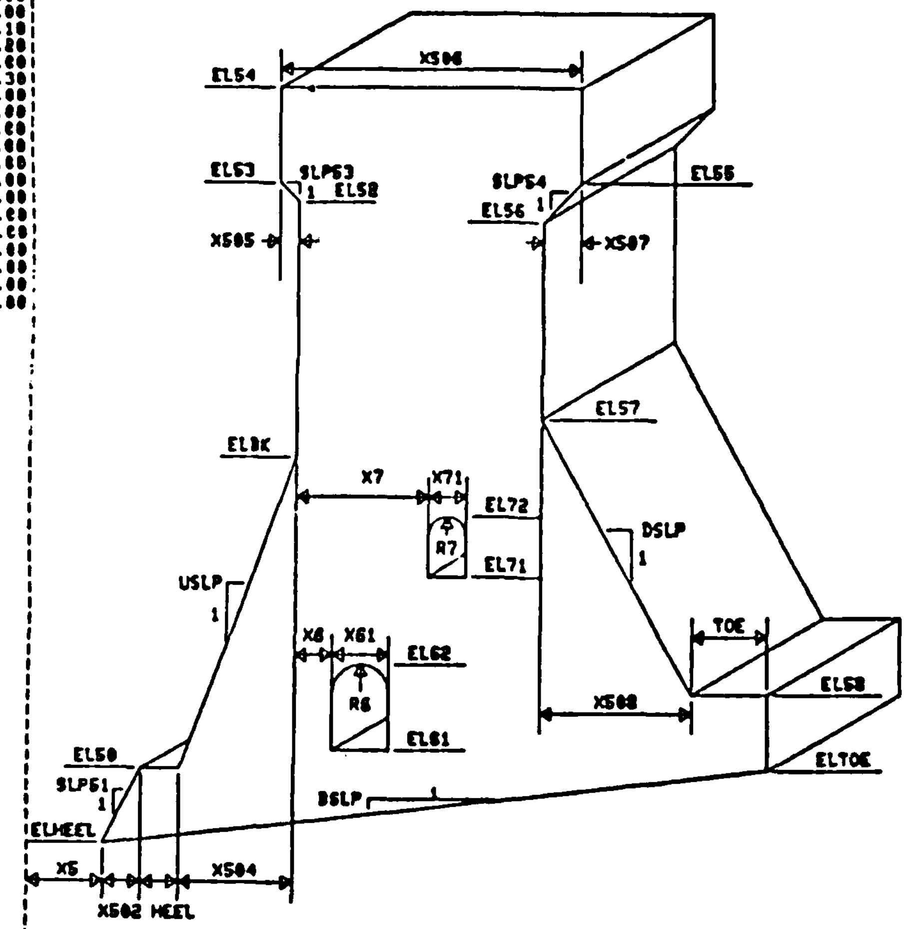
I - INPUT OR CHANGE DATA BY PLACING THE X-HAIRS OR CURSOR ON THE DESIRED LABEL IN THE PICTURE AND TYPING I.

ALL VARIABLES NOT X-ED OUT ARE 0.

P - PLOT PICTURE AND PRINT CURRENT DATA VALUES BY TYPING P.

G - SAVE DATA IN RESTART FILE AND GO BY TYPING G.

ELHEEL	=	268.00
EL50	=	283.00
SLP51	=	.10
HEEL	=	10.20
ELBK	=	283.00
USLP	=	.30
EL52	=	295.00
X505	=	-6.00
EL53	=	306.00
EL54	=	315.00
X506	=	31.00
EL55	=	335.00
X507	=	-3.00
EL56	=	256.00
EL57	=	285.00
X508	=	8.00
EL58	=	275.00
TOE	=	6.00
ELTOE	=	268.00

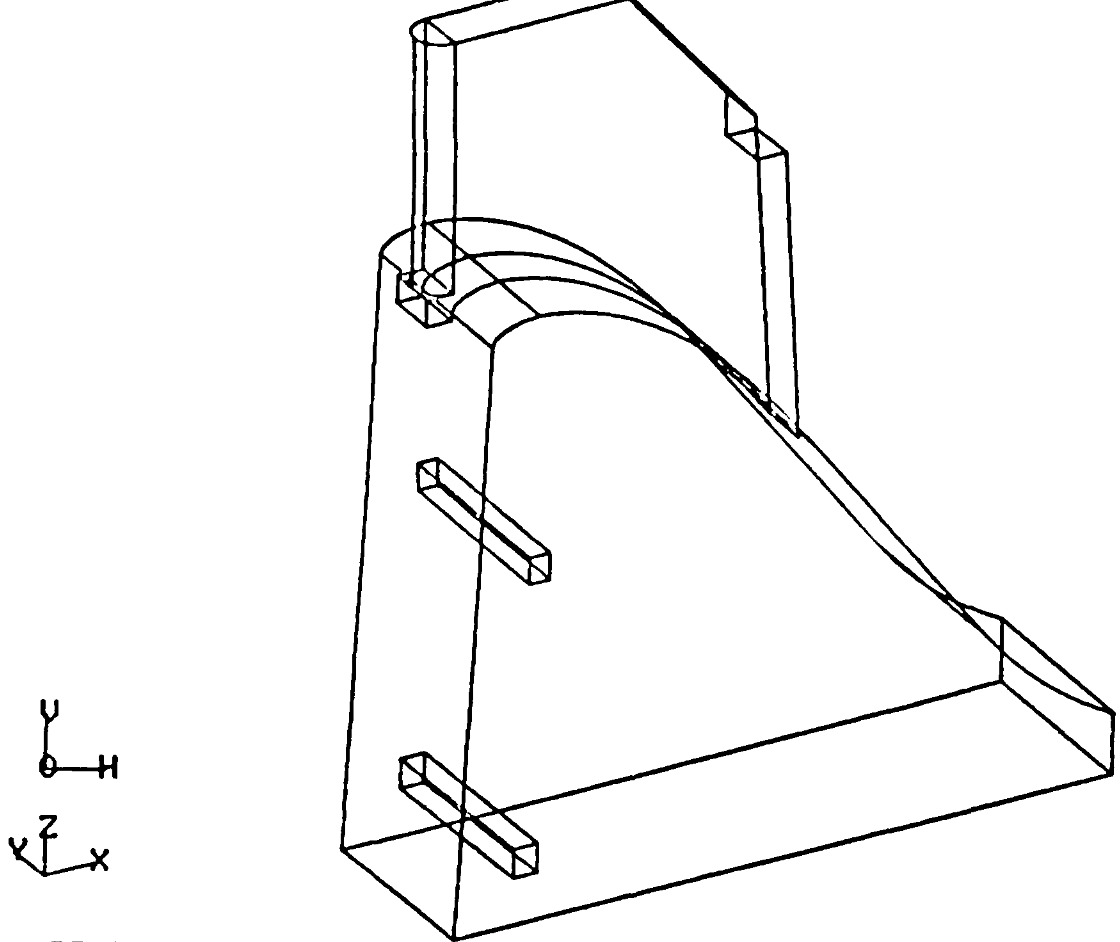


VARIABLE LIST

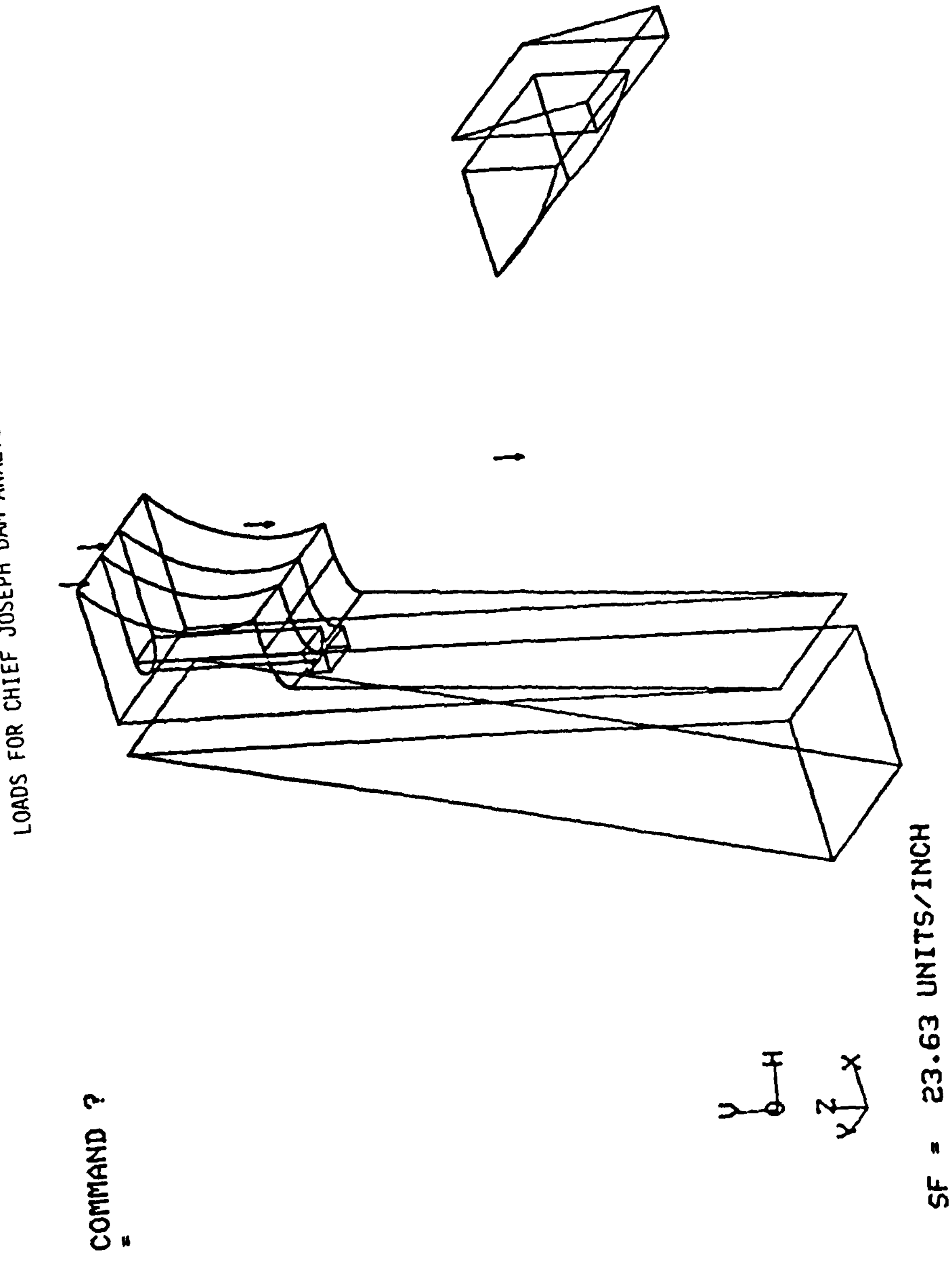
X5
X502
X504
SLP63
SLP54
DSLP
BSLP
X6
X61
EL61
EL62
R6
X7
X71
EL71
EL72
R7

GEOMETRY FOR JOSEPH DAM ANALYSIS

COMMAND ?
=GO



SF = 22.81 UNITS/INCH


LOADS FOR CHIEF JOSEPH DAM ANALYSIS
COMMAND ?
=
SF = 23.63 UNITS/INCH
Y
H
Z
X

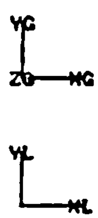
KERN PLOT FOR CHIEF JOSEPH DAM ANALYSIS

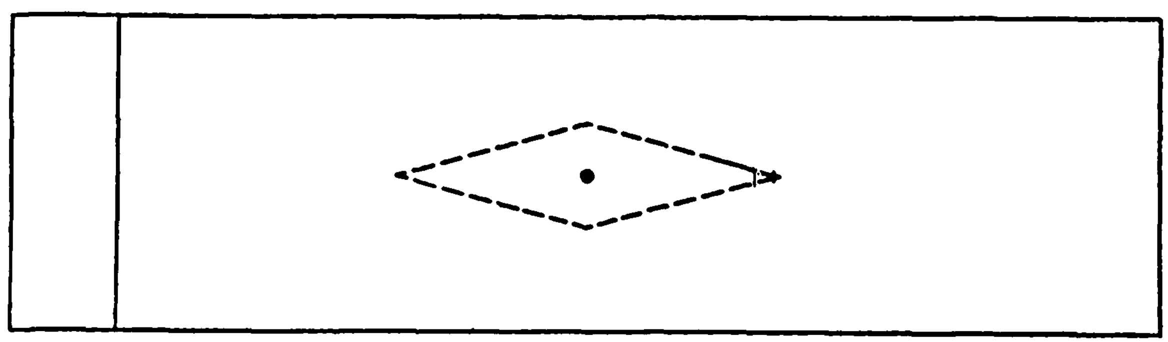
LOAD CASE II
CATEGORY = 1

ECCENTRICITY
IN LOCAL
COORDINATE SYSTEM

X ECC. = 26.81
Y ECC. = 0.00

R DIST = 0.18



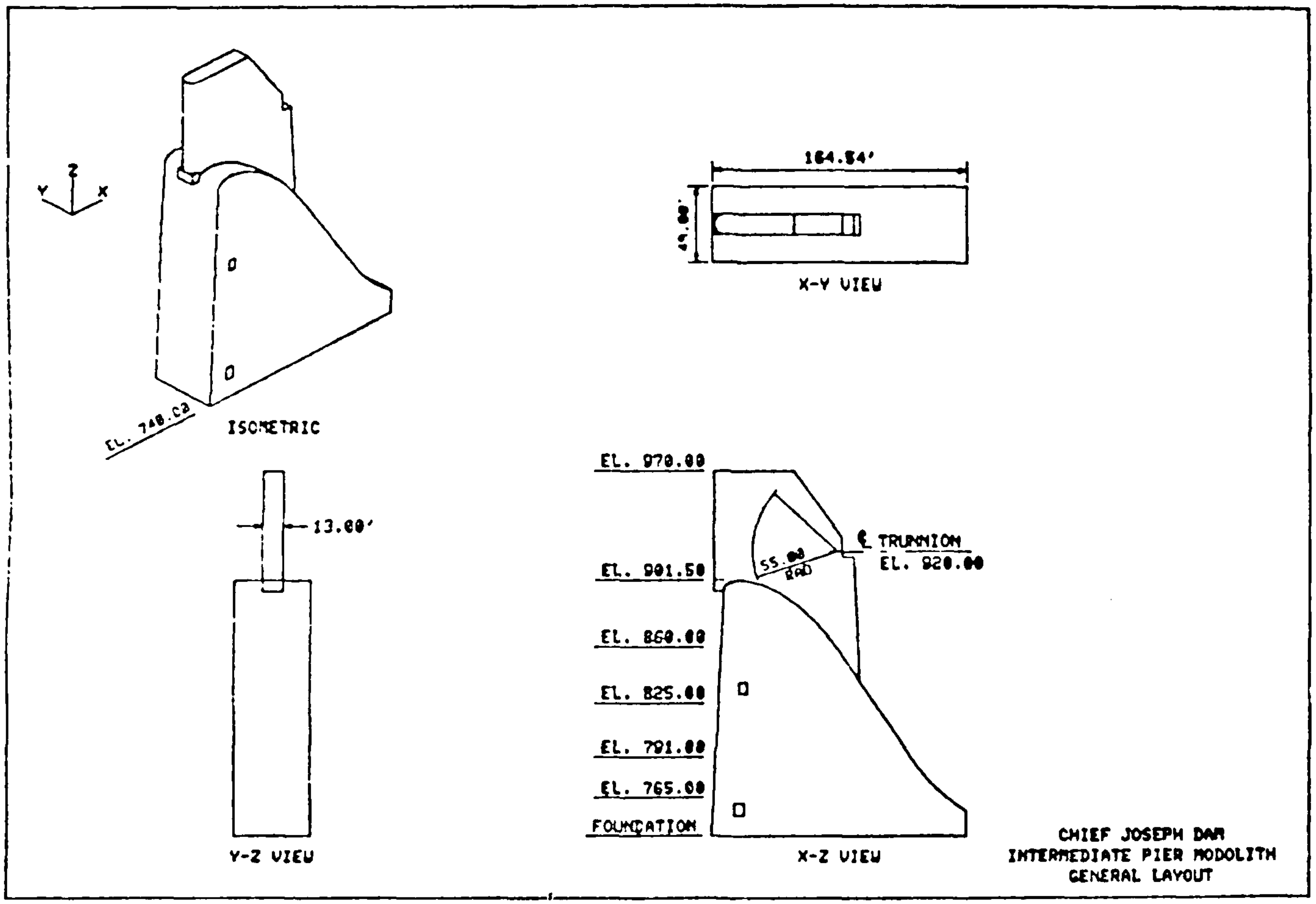


SF = 15.12 UNITS/INCH

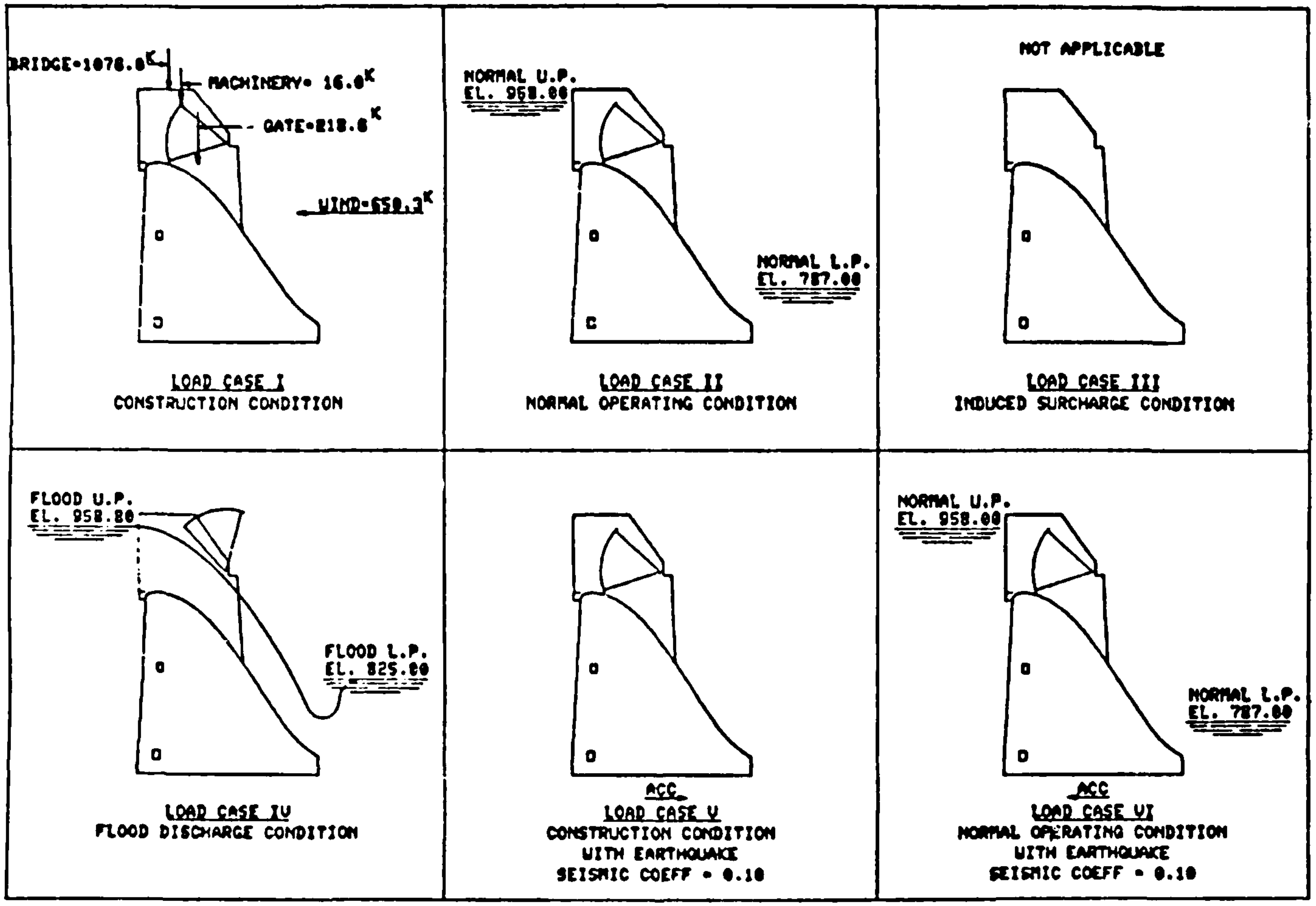
RESULTS OF CHIEF JOSEPH DAM ANALYSIS

```
                    LOAD CASE II   CATEGORY =    1

                      FINAL BASE AREA PROPERTIES

 AREA  =     8062.435      IXP  =    1613159.0      IYP  =  18189694.3
 XBAR  =      162.706      YBAR =      24.500       ZBAR =    740.000
XYANG  =        0.        ZNANG =        0.        PXANG =       0.

                  SUMMARY OF FINAL FORCES AND MOMENTS

                              --INPUT--

  FX =     69388.590       FY =         0.          FZ = -147897.971
  MX =  -3623500.3         MY =   77367950.0        MZ =  -1700020.5

                          --COMPUTED UPLIFT--

  FX =        0.           FY =         0.          FZ =   47197.355
  MX =   1156335.2         MY =   -6936213.0        MZ =       0.

                              --TOTAL--

  FX =     69388.590       FY =         0.          FZ = -100700.615
  MX =  -2467165.2         MY =   70431737.0        MZ =  -1700020.5

           FINAL IN PLANE COORDINATES AND BASE PRESSURES

 NAME              X              Y              Z           PRESSURE

 A              80.436          0.           740.000          0.280
 B              95.500          0.           740.000          2.516
 E              95.500         49.000        740.000          2.516
 F              80.436         49.000        740.000          0.280
 C             244.976          0.           740.000         24.700
 D             244.976         49.000        740.000         24.700

              FINAL SHEAR FRICTION FACTOR OF SAFETY

  PHI =        33.00    SHRSTR =        79.00   SANGLE =

            FACTOR OF SAFETY =          10.18

                       EFFECTIVE BASE = 100.0%

OVERTURNING CRITERIA SATISFIED
MAXIMUM BASE PRESSURE CRITERIA SATISFIED
SLIDING CRITERIA SATISFIED

STABILITY ACHIEVED FOR THIS LOAD CASE
```

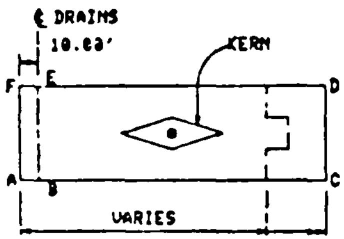
DESIGN MEMORANDUM (DM) PLATE #1 FOR CHIEF JOSEPH DAM



DESIGN MEMORANDUM (DM) PLATE #2 FOR CHIEF JOSEPH DAM



DESIGN MEMORANDUM (DM) PLATE #3 FOR CHIEF JOSEPH DAM



X-Y VIEW

ELEV.	LOAD CASE	FX	FY	FZ	r	FS	CONTACT PRESSURE/ UPLIFT PRESSURE (K/SF)					
							A	B	C	D	E	F
860	II	14706	0	-27052	.28	22.33	.273	2.014	13.18	13.18	2.014	.273
							6.126	2.649	0.	0.	2.640	6.125
825	II	27086	0	-45924	.360	16.09	.455	2.212	18.15	18.15	2.212	.455
							8.313	3.744	0.	0.	3.744	8.313
791	II	42706	0	-69804	1.013	13.08	1.501	3.051	21.07	21.07	3.051	1.501
							10.44	4.805	0.	0.	4.805	10.44
765	II	56256	0	-84584	1.033	11.86	2.706	3.868	20.22	20.22	3.868	2.706
							12.06	6.364	1.375	1.376	6.364	12.06

LEGEND:

FX - SUM OF ALL HORIZONTAL FORCES IN THE GLOBAL X-DIRECTION CONSIDERING ANY UPLIFT EFFECTS

FY - SUM OF ALL HORIZONTAL FORCES IN THE GLOBAL Y-DIRECTION CONSIDERING ANY UPLIFT EFFECTS

FZ - SUM OF ALL HORIZONTAL FORCES IN THE GLOBAL Z-DIRECTION CONSIDERING ANY UPLIFT EFFECTS

r - SHORTEST DISTANCE BETWEEN RESULTANT AND KERN CONSIDERING ANY UPLIFT EFFECTS

FS - FACTOR OF SAFETY FOR SLIDING STABILITY

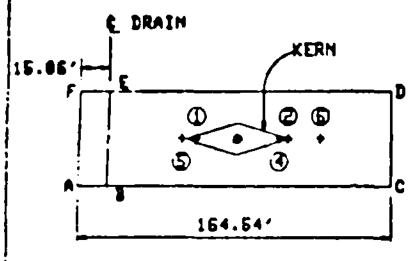
CONTACT PRESSURE - NET COMPUTED PRESSURE AT A POINT CONSIDERING THE INDICATED UPLIFT PRESSURE

UPLIFT PRESSURE - HYDROSTATIC PRESSURE AT INDICATED POINT

INTERMEDIATE PIER MONOLITH
RESULTS OF GOVERNING LOAD CASE
FOR INTERIOR SECTIONS

DESIGN MEMORANDUM (DM) PLATE #4 FOR CHIEF JOSEPH DAM

Blk 0



X-Y VIEW

LOAD CASE	FX	FY	FZ	r	FS	CONTACT PRESSURE/ UPLIFT PRESSURE (K/SF) A	B	C	D	E	F
I	-325	0	-140184	1.686	2239.	31.03	28.53	3.746	3.746	28.53	31.03
						0.	0.	0.	0.	0.	0.
II	69388	0	-100700	.176	10.12	.28	2.516	24.7	24.7	2.516	.28
						13.62	7.792	2.938	2.938	7.792	13.62
III	N.A.										
IV	64543	0	-87861	1.466	10.75	2.042	3.663	19.75	19.75	3.663	2.042
						13.68	9.111	5.313	5.313	9.111	13.68
V	-14212	0	-140184	0.	50.07	35.69	32.34	0.	0.	32.34	35.69
						0.	0.	0.	0.	0.	0.
VI	91700	0	-100700	0.	5.554	0.	0.	35.84	35.84	0.	0.
						13.62	13.62	2.938	2.938	13.62	13.62

LEGEND:

FX - SUM OF ALL HORIZONTAL FORCES IN THE GLOBAL X-DIRECTION CONSIDERING ANY UPLIFT EFFECTS
FY - SUM OF ALL HORIZONTAL FORCES IN THE GLOBAL Y-DIRECTION CONSIDERING ANY UPLIFT EFFECTS
FZ - SUM OF ALL HORIZONTAL FORCES IN THE GLOBAL Z-DIRECTION CONSIDERING ANY UPLIFT EFFECTS
r - SHORTEST DISTANCE BETWEEN RESULTANT AND KERN CONSIDERING ANY UPLIFT EFFECTS
FS - FACTOR OF SAFETY FOR SLIDING STABILITY
CONTACT PRESSURE - NET COMPUTED PRESSURE AT A POINT CONSIDERING THE INDICATED UPLIFT PRESSURE
UPLIFT PRESSURE - HYDROSTATIC PRESSURE AT INDICATED POINT

INTERMEDIATE PIER MONOLITH FOUNDATION
RESULTS OF LOAD CASES 1 TO 6

DETECTION AND MONITORING OF STRUCTURAL DEFICIENCIES
IN THE ROCK FOUNDATION OF LARGE DAMS*

AD P005690

Kalman Kovari

Swiss Federal Institute of Technology

1. Dr. Kalman Kovari is the Head of the Rock Engineering Department, Swiss Federal Institute of Technology, Zurich, Switzerland. Dr. Kovari is an acknowledged authority on applied and theoretical rock mechanics, specializing in the rock engineering aspects of dams, tunnels, and other large structures. He has analyzed rock mass stabilities as they pertain to overall structural stability in both the design phase of new construction and in the evaluation processes applied to preexisting structures. Dr. Kovari has worked with projects not only in his native Switzerland but also in other European countries and several other continents. The following is a summary of the presentation made by Dr. Kovari before the September 1985 Corps of Engineers REMR Workshop on the Stability of Large Concrete Structures on Rock Foundations at Vicksburg, Mississippi.

2. To judge the safety of a structure or understand its actual behavior, deformation measurements are usually carried out. This is the practice of all truly professional engineering organizations such as the US Army Corps of Engineers. It is applicable to almost every kind of structure, whether it be a pier supporting a bridge in Brazil, a quarry slope in Lucerne, or a subway in Munich. It cannot be emphasized strongly enough that a structure, such as a dam, and its foundation form a structural unit; consequently the behavior and safety of that dam are inseparably linked with the performance of the foundation. Moreover, it is generally recognized that it is not the average rock condition which is the cause of concern but rather the presence of specific deficiencies like weak zones, open or "healed" joints, continuous shear zones, and so on. Considering present-day numerical methods in analysis and design of dams and also the advanced construction technology and materials used, the major source of uncertainty in predicting the behavior and safety of dams, large and small, mainly resides in the actual rock mass properties and

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* A summary of Dr. Kalman Kovari's presentation by James Warriner, Geotechnical Laboratory, US Army Engineer Waterways Experiment Station.

behavior. For these reasons, it is now common (or should be) to continuously monitor the structure and its foundation throughout its service life. One of the most useful forms of monitoring is that of deformation measurement.

3. Deformation monitoring of dams and their foundations may have three basic purposes:

a. To check (confirm) normal behavior providing a measure of confidence and checking the validity of the main design assumptions.

b. To allow early recognition of deviations from the normal behavior.

c. To find the cause or causes of the unexpected behavior.

In stability evaluations, emphasis is placed on the second and, most strongly, on the third of these purposes. Therefore, it is to be expected that additional instruments and observations will be required for a structure showing unexpected behavior during its service and well after its construction. Unfortunately, the critical volume of interest, the foundation rock, is covered up by the massive structure itself. If we are lucky, there are nearby outcroppings on which we may observe areas of the rock and its characteristics, but those observations will only be indirect indicators of the rock under the structure. They are far better than nothing at all but the rock defects causing erratic behavior are highly localized and must be examined and instrumented directly.

4. Generally, only a very few displacement vector components from selected points on the structure or in boreholes will be achieved by standard instrumentation programs. This sparsity of data, when compared with the wealth of information provided by sophisticated analysis methods like the finite element, is a severe restriction on the meaningful application of those methods. Such limited measurements may be referred to as "point-wise observations." As long as the size and direction of the observed displacements are acceptably close to that of the predicted ones the "point-wise observations" fulfill their purpose. However, in the instances of exceptional behavior, the situation is completely different. Hidden details like particular joints or weak zones that are relatively distant from the points of observation might play a crucial role. The concept of "line-wise observation," along with high precision instrumentation, will supply a larger and more meaningful amount of information and, thus, may provide the key to proper understanding of complex

geotechnical situations, their causes and effects. "Line-wise observation" means measuring the distribution of a deformation quantity along a line. Borehole inclinometer devices that measure inclinations while traversing the borehole are examples of current "line-wise observation" technology. Anchored extensiometers, whether single element or multiple element, are examples of "point-wise observation" in that their measurement points are the fixed anchors in the borehole.

5. Consider a borehole as a measuring line inside a rock mass. If the mass deforms, then the line will also deform. The more we know about those measuring line deformations, the more we know about the deformations of the volume of the medium surrounding the line. We may affix points along the measuring line and measure their movements relative to the end of that line. That is then the extensiometer concept. However, each of those separate measurements of movement incorporates all deformations in the intervening length. The pointwise extensiometer is essentially an integrated measurement of displacement only, and limited in scope as well. More meaningful, both in terms of localization of measurements and in terms of available quantities of data points, would be a measurement scheme directed towards the differential of axial displacements which is axial strain. Differential measurements of axial strain within a measuring line are achievable using modern instrumentation design and fabrication. A device which uses the concept of differential measurements of strain within solid media is the borehole micrometer.

6. The borehole micrometer is a 1 meter long probe with a spherical head at each end. The device is lowered into a borehole to specific measurement locations at each of which is a pair of conical seats 1 meter apart fixed within the casing and through grout to the rock mass. When the probe is made to seat its spherical end pieces in the conical seats, an internal transducer measures the actual separation of those seats to the nearest 3 micrometers. Considering the nominal base length over which the differential is measured is 1 meter, then the axial strain of the borehole (and therefore the strain of the surrounding rock) is measured with an accuracy of 3×10^{-6}. This is sufficient accuracy for stress measurements to the nearest 150 psi in typical rocks, or for detection of very minute rock joint openings as overlying structures are loaded.

7. Differential borehole axial strain measurements using the borehole micrometer device have been made beneath many types of structures and have been used to locate individual joints and weak zones which were critically involved in exceptional behavior of those structures.

8. At the Kolnbrein arch dam in the Austrian Alps, subsurface rock strain measurements indicated not only a change in deformation magnitudes with increasing water levels, but also a major redistribution of stresses in the foundation rock. It was concluded that there was an unexpected reduction of the compression zone in rock beneath the upstream toe of the dam caused by a simultaneous decrease in the load transfer surface at the base and a change in inclination of the dam thrust to increase shear forces relative to normal forces.

9. The Albigna gravity dam in the Swiss Alps demonstrated cracks in several monoliths that propagated down into the rock foundation. By the use of subsurface rock strain measurements, a pair of active rock joints were identified that were opening and closing by as much as 3.86 mm per meter with changes in water level. The joints were later found to daylight on the reservoir bottom near the toe of the dam. Stress field interpretations of the borehole strain data, together with verification of no other joints opening and closing, allowed confirmation of the dam's stability and close control of remedial grouting to stop the underseepage through the pair of opening joints.

10. In his presentation Dr. Kovari briefly outlined some modern considerations in the evaluation of behavior and safety of large structures built on rock. He also described the concepts of measuring strains directly in in situ rock as opposed to simply measuring displacements. Additionally, the instrumentation means for measuring those in situ strains were described.

AD-P005691

COMMENTS ON A PROPOSED INVESTIGATION OF LATERAL EARTH PRESSURES EXERTED BY BACKFILLS

Wayne Clough
Virginia Polytechnic Institute and State University

1. The rehabilitation of navigation structures requires consideration of the lateral earth pressures acting upon the structure. Typical assumptions for earth pressure loadings usually follow patterns dictated by classical theories. For example, at-rest pressures with a simple triangular distribution are often used. Although it has been known for some time that actual earth pressure distributions are more complex than this, in many cases it is expedient to use the simplest approach, and experience suggests that this approach is usually conservative. However, the problem of the navigation structure is one where the simplest assumption is not likely the best one, and because the assumed lateral pressures significantly impact the economics of rehabilitation, the problem deserves study under the Repair, Evaluation, Maintenance and Rehabilitation (REMR) research program.

2. When considering a navigation structure, there are many factors that potentially cause the lateral earth pressures to deviate from conventional assumptions:

a. Compaction effects and long-term creep in the backfill.

b. Cyclic movement of the structure walls due to alternating water levels and temperature effects.

c. Structural shapes that deviate from that of the simple retaining wall.

d. Backfills that are placed in cuts of limited extent in natural materials adjacent to the structure.

Depending upon which combination of the factors dominates any particular problem, the lateral earth pressure resultant may be either larger or smaller than those of the at-rest type assumption. In many cases, the earth pressure distribution is also likely to deviate from normal assumptions, thus affecting deliberations about overturning and moment distribution.

ENGINEERING TECHNICAL LETTER, "STABILITY CRITERIA FOR REHABILITATION OF NAVIGATION CONCRETE STRUCTURES"

M. F. Lee

Directorate of Engineering and Construction
Office, Chief of Engineers, US Army

SUMMARY

A copy of the draft Engineering Technical Letter (ETL) was handed out to attendees. Mr. Lee talked about the purpose, background, and contents of the ETL. The goal was to get the ETL published by January 1986. Pittsburgh District personnel were still working on the section of the ETL on rock anchors at the time of the Workshop. The draft ETL that was distributed at the Workshop is provided as Appendix B. This draft ETL is continuing to be revised and should not be used without HQUSACE (OCE) consent.

3. Achieving the objective of an improved earth pressure prediction technology will require studies along several different lines with complementary ends:

a. Measurements of lateral backfill pressures.

b. Studies of lateral pressures using reasonably sized laboratory model systems.

c. Analytical studies of full-sized navigation structures using modern finite element technology.

The first of these is needed to develop confidence in the results of the model and analytical investigations. No reasonable engineer would undertake major changes in the assessment procedures for earth pressures based only upon laboratory or analytical methods. Measurements of loads acting on prototype structures is a key ingredient in developing confidence in any new findings that might be generated. The second task, model studies, will be important since they can allow careful measurement of earth pressures in a controlled environment. In addition to the question of lateral pressures, per se, they offer the opportunity for full-scale testing of any devices proposed to measure pressures in the backfill or stresses acting on the structure. Finally, the finite element analyses described for the third task are useful because they can generate information efficiently on large numbers of relative parametric effects. The analyses can be calibrated by the results of the other two phases of the overall study program. The results of the finite element analysis will be particularly useful because they can be molded into a simplified design method through charts and personal computer based programs.

4. The main focus of this document concerns the first of the proposed study tasks, namely, measurement of earth pressures acting on existing structures. The following paragraphs are devoted to a discussion of the issues associated with this topic.

Measuring Earth Pressures Acting on Existing Structures

5. Relatively little attention has been directed to this general subject area, probably because the profession has been more concerned with building new structures instead of rehabilitation. Fortunately, it is not necessary to

create an entirely new technology for the desired purpose, but rather to accept existing procedures to the task at hand.

6. There are at least three different approaches that can be applied:

a. Perform tests in the backfill near the structure with "active" in situ test instruments.

b. Place "passive" instruments in the backfill.

c. Identify those cases where earth pressure cells are installed on navigation structures, and directly measure lateral pressures.

Backfill Tests with "Active" In Situ Devices

7. In this text, the term "active" in situ device is meant to apply to a device that is inserted into the backfill, and which expands or presses against the soil in the test. Instruments in this category for which we have a good experience base include the pressuremeter and the Marchetti dilatometer.

8. The idea for this work task would be to utilize present day instruments, or slight modifications thereof, to measure the lateral stress in the backfill of many retaining structures. Assuming that this is done near enough to the structure, the soil pressures should be the same as those acting on the structure. Because of the recent advances in in situ testing technology, it would appear that this task can be achieved in many cases using available or slightly modified instruments.

9. Active in situ devices for which there is good experience in measuring lateral stresses in soils include various types of pressuremeters, and the Marchetti dilatometer (5)*. It may also be possible to include in this category the concept of hydraulic fracture, but this procedure would have, at most, only a limited range of applicability for the backfill situation.

10. Pressuremeter technology has come a long way since the introduction of the original Menard pressuremeter in 1955. Several new versions of the pressuremeter have been introduced, and considerable improvements have been made

* Numbers refer to References found at the end of this paper.

in the manner in which measurements are made during the test, leading to both better quality test data, as well as additional information not available before. The basic Menard pressuremeter test involves preboring a hole, inserting the probe into the hole, and expanding the probe while measuring the volume of hole and the pressure applied to the probe membrane. Unless special measures are taken, this approach has historically been found to be lacking in accuracy and repeatability in determining lateral stresses (8). The self-boring pressuremeter, introduced in the early 1970's, has been found to lead to more reliable lateral stress measurements (1, 4, 9). It would appear to have potential for the application anticipated for this work and warrants additional discussion.

11. It is not possible to review the many papers that have been published on the self-boring pressuremeter. Useful summaries are given in references 1, 2, 4, 7 and 9. Some key characteristics are as follows:

a. Is well adapted to the use of automatic data acquisition systems.

b. Can be used to measure the lateral stress in several directions in a soil in one test.

c. Interpretation of the results for lateral stress is simple.

d. Test results can be used to determine soil strength and stiffness in addition to lateral stress.

Limitations with the self-boring instrument are primarily related to problems with probe advance in very stiff clays and gravelly soils. Improvements have been made in these areas with the introduction of new drilling techniques and the use of directed jetting as an alternative to drilling the probe in place. However, for some cases, such as gravelly soils, self-boring is impossible. Such conditions call for the preboring or driving open the hole for the pressuremeter. If preboring or driving is done with care, and the tests are designed properly, in many instances reasonable measurements can be obtained. To be able to derive the degree of confidence that is needed, additional testing would be useful to check the accuracy of pressuremeter measurements when preboring or driving is used to open the hole. This will be possible if the in situ probe evaluation is combined with the model testing effort as is described subsequently.

12. Although the self-boring pressuremeter is probably the best method for determining in situ stress in soil, other techniques are showing promise, particularly in difficult soil conditions. Of the new methods, most of the experience has been with the Marchetti dilatometer. This device relies on empirical procedures to obtain lateral stress, since the probe disturbs the ground as it is inserted. Recent experience in measuring lateral stresses in gravelly sands at Lock and Dam 26 using the dilatometer showed that this device gave values which were more consistent than those obtained by other techniques (3). The dilatometer is attractive because it is very rugged and strong. Because most of the research with the dilatometer has been in uniform soils, research is needed to determine exactly what its capabilities are in the types of material used for structural backfills.

13. To this writer's knowledge, there have only been two cases where active type in situ probes have been used to measure lateral earth pressures in an environment such as a retaining structure backfill (2, 3). Although both of these test programs were relatively successful, the experience base is limited. Before field testing is done for the REMR program, controlled tests should be conducted using the most promising of the in situ probes. Such a program of tests could be linked to the model tests that are recommended as a part of the larger investigation of lateral pressures. If the model tests are of suitable size, in situ tests could be performed directly in the backfill of the model and checked against known pressure conditions.

Pressure Measurements in Backfills Using "Passive" Instruments

14. Passive type instruments are those that are used to measure lateral stresses in soils without any movement of a membrane or any other part. These devices are inserted into the ground and remain in place until an equilibrium is reached. The amount of time required before the equilibrium is reached varies depending upon the type of soil and the method used for insertion. Examples of the passive type of device are the Gloetzl cell (7), the Camkometer (9), and the lateral-stress cone (6).

15. The Gloetzl cell and the lateral-stress cone are inserted by pushing them into the ground. This presents a problem in stiff or gravelly soils, and

there is little that can be done to overcome it. Because of this limitation, the Gloetzl cell is used only in soft to medium clays. The lateral-stress cone, which has only recently been introduced, can be used in more difficult conditions than the Gloetzl cell, since it has a torpedo like shape, and it can be designed to withstand high thrust loads. This device has an added advantage in that the thrust readings during insertion can be used in empirical correlations to estimate soil properties other than strength. However, experience with the lateral-stress cone is limited, and further testing is needed before it can be used with confidence.

16. The Camkometer may be thought of as a self-boring pressuremeter without a membrane or any capability to load the soil. It has load cells to measure the pressure acting against the sides of the probe. The Camkometer has not found much use in geotechnical engineering because most engineers would prefer to use the self boring pressuremeter inasmuch as it can be used to get information on both lateral stresses and the soil strength.

17. Of all of the passive type instruments, the lateral-stress cone appears to be the best candidate for use in the REMR research program. Should it be included as a candidate, it should be integrated into the test effort in the model testing phase so that it can be evaluated directly against the active type probes. This is particularly important for the lateral-stress cone since it is a very recent development and has not been subjected to extensive scrutiny.

Earth Pressure Cell Measurements

18. Earth pressure cells embedded in the walls of retaining structures have been used on a number of occasions to measure lateral stresses. Of course, to use earth pressure cells, they must be installed prior to backfilling. Thus, earth pressure cells have little application in specific cases of rehabilitation since they have been rarely installed. However, there are some instances where earth pressure cells can find application. First, in the few cases where earth pressure cells have been installed in navigation structures (e.g., Port Allen and Old River Locks), the cells could be read again to assess long-term earth pressures, assuming that the cells are still operative. Second, in

the future where a new structure is to be built, earth pressure cells could be installed with a monitoring program designed to capture those data which are deemed useful to the issue of lateral loading. These data could be particularly useful in establishing a prototype baseline which could complement the information determined by the other techniques.

19. Finally, if it is determined that the earth pressure cells on some existing structure are in working order, this would offer an opportunity to field test the best of the in situ probes. The lateral pressures measured by the probes could be checked against the values recorded by the earth pressure cells.

General Comments

20. Measurement of the earth pressures acting on the walls of navigation structures is of value in view of the complex nature of the problem and its importance to the issue of how to rehabilitate older units. Direct measurement of the pressures is only possible where earth pressure cells were fortuitously installed during construction. Unfortunately, this has been done in few cases, and even fewer of these are still operable. This leads to the need for determining the pressures by means of measurements of the lateral stresses in the backfills near the structures. Many instruments have been proposed for the purpose of determining lateral stresses in soils in an at-rest state. Of these, the pressuremeter (self-boring and nonself-boring), the dilatometer, and the lateral-stress cone would appear to have potential for addressing the problem of measuring pressures in retaining structure backfills.

21. If the soils in the backfills are not gravelly, the self-boring pressuremeter would appear to provide one of the best options. In softer soils and sands, a self-boring advance could readily be used. In stiffer clays and silts, the majority of the advance could be achieved with preboring, and self-boring used only for the final stages before the test depth. Alternative procedures using roller bits and high-pressure jetting are in the process of development. In gravelly soils, preboring and/or roller bit drilling would be required, although in such instances, disturbance may be a problem. It is also possible that the conventional pressuremeter may suffice in cases where

gravels and cobbles are present. Testing of the different approaches in a laboratory model test apparatus can help resolve the issue of how much disturbance is likely to occur.

22. Both of the other candidate probes, the dilatometer and the lateral-stress cone, involve insertion by pushing. This inherently leads to a certain degree of disturbance, and thus, these methods are theoretically not as accurate as the self-boring pressuremeter. However, in more difficult environments, such as gravelly soils, the dilatometer or the lateral-stress cone may prove advantageous. Testing in a laboratory environment is needed to help sort out the accuracy that can be achieved in this task.

23. The program of checking out candidate in situ testing probes should be coordinated with reasonably sized model tests of an instrumented retaining structure. In this way, the probes can be used directly to determine pressures in the model backfill, and the data can be compared to the pressures measured that act on the wall itself. These types of tests will also allow for the possibility of modification of the probes to enhance their capability to measure backfill pressures.

24. Field trials of the candidate probes might well be carried out at a site where a navigation structure has operating earth pressure cells. This will provide a prototype check on the capability of any device to measure the earth pressures acting on the structure.

25. Finally, it is realized that in many cases, the backfills of navigation structures are composed of rocky materials, and such circumstances are probably beyond the capability of in situ stress measurement technology. In this type of environment, not only is it difficult to introduce a probe into the ground, but also the point to point contact of the rock fragments makes it almost impossible to find a location where stresses can be measured. However, it is the opinion of this writer that a great deal can be learned about the earth pressures acting on navigation structures by making measurements for those cases where conditions are suitable. Using reasonable judgement, these measurements can provide key information for the purpose of extrapolating to problems where earth pressures cannot be measured.

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SHEAR STRENGTH SELECTION PROCEDURES AND THE USE OF THESE PARAMETERS FOR EVALUATING THE STABILITY OF EXISTING CONCRETE STRUCTURES

CHAIRMAN: Glenn Nicholson, WESGR-M
RECORDER: Hari N. Singh, NCDED-G

MEMBERS : Marve Simmons, ORDN-G
Frank Weaver, LMVED-G
Lavane Dempsey, NCDL-S
Jack Berezniak, NABEN-F
Lloyd Oliver, SAMEN-DC
Charles Canning, ORDED-G
John Speaker, ORLED-D
John Cribar, ORPED-DM
Stuart Long, ORPED-GG
Larry Schlaht, MROED
Garrett Johnson, NPSEN-DB
Charles Dowding, Northwestern University

Goals of the Working Group

1. Summarize the present procedures for determining parameters for the purpose of computing the driving and resisting forces.

2. Identify shortfalls in the present procedures for the purpose of evaluating the stability of existing concrete structures on rock.

3. Recommend potential solutions to overcoming the identified shortfalls with emphasis on R&D plans for addressing the problems.

Introduction

4. Many Corps employees feel that conservative analysis procedures and shear strength parameter selection procedures are the chief reasons that some aging Corps structures have calculated stability safety factors less than desirable. Available guidance does not provde any information as to the material parameters for which the required factor of safety is related. If residual strengths are used, the factor of safety should possibly be lower than 2.0. Adding additional stability to an existing structure is costly and, therefore, should not be done unless it is truly needed. The calculated safety factor against sliding of a concrete structure on rock is very sensitive to the shear strength parameters selected for the analysis. This report presents the discussion and findings of the working group on the selection of shear strength parameters.

Present Procedures

5. The most common approach for obtaining shear strength parameters for analysis of stability of structures on rock involves several steps:

a. Determination of potential failure planes.

b. Retrieving undisturbed samples from the potential failure zone.

c. Testing specimens prepared from undisturbed samples.

d. Selection of failure criteria.

e. Evaluation of shear strength parameters to be used in design.

This approach of obtaining strength parameters is known as the Testing Approach and it is used by the Corps of Engineers in some form. There are two other approaches known as the Rational Approach and the Empirical Approach for evaluation of shear strength parameters. These have not been used by the Corps. Detailed discussions on these approaches are given elsewhere in this report.

6. Although the Corps of Engineers uses the Testing Approach, there are no guidelines regarding selection of appropriate test samples, selection of failure criteria, and design shear strength parameters. Guidance on how to select failure criteria has not been published and consequently has led to a wide variation in choice of design parameters. For example, some divisions use peak strength parameters; whereas, others are using residual strength parameters and still others use some intermediate value between peak and residual. Definitive guidance is needed on when peak strength parameters or residual strength parameters are to be used, or when some value between the two should be used and on how this value is to be selected. Because of the lack of guidelines on selection of test samples, shear strength parameters are obtained by (a) testing cores of intact rock and (b) determining the friction between sawed unweathered rock surfaces prepared by a diamond saw. No regards are given to the rock mass discontinuities even though they may form part of the potential failure plane. The strength parameters corresponding to intact rock specimen and flat sawn surfaces are designated as upper bound and lower bound strength parameters, respectively. Effects of asperities and weathering conditions of joints are often ignored because of the difficulty and uncertainty of defining them.

7. The shear strength parameters for the rock-concrete interface are sometimes determined by testing a specimen prepared by casting grout on a rock specimen. There is no standard for preparing specimens for testing concrete-rock interface strength. In some Corps laboratories, specimens are prepared by casting grout on rough sawn rock surfaces. In others, fractured rock surfaces are used. The different methods of preparing the specimens results in considerable variations in strength parameters for the same type of rock and grout.

8. Triaxial tests are normally used for testing intact rock, and strength parameters are obtained by drawing Mohr Circles (Mohr-Coulomb failure criteria). The friction between precut surfaces are determined by Direct Shear Tests. Discontinuity strength parameters are rarely determined. Strength of gouge materials are often determined from fabricated specimens. In the case where gouge materials cannot be obtained, artificial gouge is prepared by crushing the rock that forms the joint walls. Test specimens are then prepared by compacting the artificial gouge at the desired compacting effort and moisture content based on the judgment of the engineer or geologist in charge of the work. Soil direct shear testing devices are normally used to test gouge specimens. It is sometimes possible to determine the strength of gouge materials from sampled specimens or large-scale in situ tests.

9. It was the consensus of the working group that the exploration and selection of representative samples for determining strength parameters are not consistently done. Mostly inexperienced Corps personnel, who have little idea of the sensitivity of the strength parameters to sample disturbance, make decisions in the field as to drilling operation and sample selection. When exploration is done by contract, the situation generally becomes worse due to the tendency to cut corners in order to produce more in less time. Contractors seldom perform detailed investigations in selecting materials for testing. Gouge materials are seldom recovered for testing by some contractors.

Shortfalls in the Present Procedure

Lack of Criteria Regarding Shear Strength Parameter Selection

10. There is a complete lack of reference materials in Engineering Manuals concerning the selection of shear strength parameters for analyzing the sliding stability of structures founded on rock. Strength parameters for various critical stages (peak strength, ultimate strength, and residual strength) are normally obtained by performing tests or samples, but there are no guidelines for choosing a failure criteria or selecting which strength parameters should be used to calculate the factor of safety.

11. Each individual Corps division has its own failure criteria based on their past experience. For example, in Ohio River Division (ORD) residual strength are often used on the basis of site specific conditions. Working group members from ORD reported that investigations of the failures of the Uniontown and the Cannelton Cofferdams in 1971 and 1974, respectively, indicated that sliding along presheared clay seams was responsible for both the failures. Because of the preshearing, the gouge materials had already exceeded their peak strength and had reached the neighborhood of residual strength. From tests results of such materials from several locations in the Ohio River Division, it has been concluded that gouge materials in discontinuities of the entire Ohio River Valley are presheared; therefore, it is appropriate to consider residual strength parameters for evaluating sliding stability of structure founded on such discontinuities. The shortfall of this procedure is that if the clay-filled joint is not daylighted or connected with a low-angle fault downstream of the structure, the clay-filled joint may not be the potential failure plane. Therefore, in the opinion of some working group members, it is not appropriate to use residual strength parameters unless the potential failure planes have been identified, through an investigation, as clay seams which are daylighted or connected with some low-angle fault downstream. In both cases of the failures (Uniontown and Cannelton) reported by ORD, these failure planes existed.

Factor of Safety

12. Some members of the group felt that using the same factor of safety for various kinds of materials is a shortfall in our present shear selection and design process. Higher factors of safety should be used for rock, and comparatively lower factors of safety should be used for soils. All members, however, did not agree with this idea. The strength of a rock mass is usually controlled by joints and their filling materials and thus a rock mass may be less homogeneous than a soil. Hence it is not advisable to apply different factors of safety to rock and soil. All of the members agreed that acceptable

factors of safety should be lowered (less than the currently acceptable value of 2) if residual strength parameters are used in stability analysis.

Discontinuities, Asperities and Loading Conditions

13. With the exception of major new projects, the orientation of discontinuities with respect to applied forces, asperities and their orientation in discontinuities are generally ignored in the shear strength selection process. Defining orientations of discontinuities and asperities is a very definite key in establishing strength parameters for design or evaluation of existing structures. Shear parameters selected disregarding the above factors are not appropriate for evaluating sliding stability.

14. The current practice of determining strength parameters by testing specimens of intact rock for upper bound strength and testing saw-cut surfaces for friction to determine lower bound strength is meaningless for the purpose of evaluating sliding along a discontinuity. It is obvious that at normal stresses ranging from 1 to 20 tsf (stress range normally encountered in the foundations of Corps structures), the shear strength of joints will be much lower than the strength of unweathered intact rock forming the joint walls (Corps upper bound strength); on the other hand, the shear strength exhibited by the flat diamond cut surface of the rock (Corps lower bound strength) will be much lower than the actual strength of the joints featuring irregularities. Therefore, stability analyses performed on the basis of these upper bound and lower bound strengths have no relation whatsoever with the actual stability of the structure. The results of the analyses based on lower bound strength are judged to be controlling and, therefore, our evaluations underestimate the stability of a structure against sliding. Most of the hydraulic structure instability problems in North Central Division are, in part, the results of this shear strength selection procedure. Members from ORD reported that Uniontown and Cannelton Cofferdams, which failed in sliding along weak clay seams, were designed on the basis of shear strength of intact rock.

Lack of Definition for τ at Salient Strain Values

15. Many group members felt that there is a lack of criteria to adequately define ultimate and residual strength parameters. Selecting these parameters becomes subjective, because it depends upon the judgment of the person making the determination. To eliminate the subjectivity, criteria should be established for these parameters.

Deformability of Rock Mass

16. Present stability analysis does not provide information regarding movements of structures in lateral as well as vertical direction. Many group members expressed their concern for a lack of information on this type of movement and cited some examples where structures are standing intact but are inoperable due to lateral movements. Therefore, they felt that shear strength parameters are not the only material parameters needed to fully analyze concrete structures on rock; deformability should also be given consideration. Lateral movements of structures (especially of lockwalls) must be limited to a value such that associated components of the structures such as gates, valves, etc., remain operable and the structures remain functional.

Exploration and Sampling

17. It was the general consensus of the group that Corps exploration and sampling procedures on some projects are not satisfactory. In some cases, explorations are supervised by inexperienced personnel with little or no knowledge of shear strength selection procedures. When the exploration work is contracted, the situation generally becomes worse because of the tendency to avoid detailed investigation in selecting samples for testing. When samples are received by the testing laboratory, there is generally no way to verify their appropriateness as to the degree of disturbance and location along the potential failure plane. Some members of the group who had occasions to observe exploration operations, reported that in some cases, gouge materials in discontinuities obtained with drill core are cleaned off before placing cores in core boxes. Thus, testing laboratories receive core samples for testing which are not representative of the actual condition of the rock mass.

18. Current practice includes obtaining NX cores which are only 2-1/8 in. in diameter. With such a small-diameter specimen, it is not possible to have a sample which includes all the rock joint variables controlling the strength parameters. Therefore, it is impossible to determine joint asperities and their effects on strength parameters using NX core samples. Large samples are essential to evaluate strength parameters realistically. Members of the group, especially from ORD, felt that at least 4- to 6-in.-diameter cores should be tested.

19. With our present system of obtaining samples by drilling operations, it is not always possible to obtain undisturbed samples of weak seams. Gouge materials are disturbed due to the spinning effect of the drilling operations and the washing action of the drilling fluid. Therefore, in actuality, we test disturbed samples, and as such we obtain strength parameters which are lower than the in situ values. Sometimes samples of gouge material can be obtained with wire line or triple tube coring equipment. With deeply buried seams this equipment is the best choice for obtaining gouge material. It was reported by some members that when gouge materials cannot be recovered by drilling, they are prepared artificially by grinding rock cores. The strength parameters of gouge determined by this method do not represent the strength of actual gouge, but some members felt it provided a good estimate of the natural gouge strength.

Testing Procedures and their Shortfalls

20. Direct Shear Test. Various laboratory test methods and their advantages and shortfalls were discussed in detail. The Direct Shear Test, the most commonly used test, has the following shortfalls:

a. Vertical stresses are not uniformly distributed on the failure surface.

b. Pore pressure in clay-filled joint cannot be controlled.

c. Variation in strength parameters is obtained for the same material tested in different shear test devices.

d. Principal stresses are not known.

The advantages of this method are that the tests are simple to perform and they permit application of load parallel to the failure planes.

[illegible]. Triaxial Tests. This method is rarely used for determining strength parameters of discontinuities. It requires special specimen preparation with the orientation of joints inclined between 25° to 40° to the major principal stress. End capping introduces lateral restraints.

[illegible] . In Situ Tests. In situ tests provide strength parameters closer to actual values than those obtained from laboratory tests. However, this method has two major shortfalls. First, tests are very expensive and time consuming, and second, in many cases the specimens which should be tested are inaccessible. Some group members felt that in situ borehole index tests provide valuable information about weak seams and, therefore, should be performed on a routine basis.

[illegible]. It was the general consensus of the group that of the three types of tests discussed above, the Direct Shear Test method is the simplest and most adaptable to loading conditions. Some members of the group, however, felt that research is needed to eliminate or minimize the shortfalls of this method. After some discussion, the majority of members were of the opinion that such research has been conducted all over the world on improvement of testing devices including the direct shear testing device and we should use the state-of-the-art testing device. The group decided that steps should be taken: (a) to standardize the direct shear testing device to eliminate variations from one device to another, (b) to standardize special tests, and (c) to develop a procedure for borehole index tests for thin clay fills.

Potential Solutions to Overcoming Shortfalls

[illegible]. It was the general consensus of the group that in order to determine the reliable shear strength for discontinuities and the structure-rock interface needed for sliding stability analysis, it is essential to (a) improve our current procedure of sampling and testing, (b) establish realistic failure criteria, and (c) review and validate methods for [illegible] strength determination other than the method (Testing Method) currently used by the Corps. The group discussed the Rational Method and Empirical Method developed by Barton, and Archambault and Ladanyi.

[illegible]. Rational Approach to Determine Shear Strength. This method uses the basic friction angle, ϕb, and asperities angle, i, to predict the shear strength ($\tau = \sigma \tan (\phi b + i)$). The ϕb is obtained [illegible] mineral constitution of the rock or by friction testing of saw-cut surfaces. Asperity angle, i, is the inclination of asperities from the horizontal in the direction of applied load. The shortfalls of this method are difficulties in evaluation of asperity angles and value of ϕb for weathered joints. At present, there is no recognized procedure for determining asperity angles. The group recommended research be undertaken for: (a) developing procedures for assessing reliable values of asperity angles, (b) evaluating ϕb for weathered joints, and (c) strength parameters based on classification of joints.

[illegible]. Empirical Approach. Two empirical approaches (Archambault-Ladanyi Approach and Barton Approach) are currently available for determining shear parameters. The Corps of Engineers does not use these approaches to evaluate shear strength parameters; however, there is potential for using these approaches for small projects where the high cost of testing cannot be justified. Members of the group felt that it was worthwhile to discuss the merits and shortfalls of these approaches and recommend their validation by the Corps. The Archambault-Ladanyi Approach needs input from tests conducted on only one or two specimens. Shear strength over a large range of stress conditions is extrapolated from data on a small number of tests. The shortfalls of the method are that the tests conducted have to be in direct shear mode and conducted on large blocks. The Barton Approach is based on joint conditions and classifications of walls of joints. The main shortfall of this approach is that the results are very subjective, because there is a lot of judgment involved in classifying joints. Two other factors, compressibility and dilation angle, needed to evaluate the strength parameters are difficult to determine. Despite the shortfalls, group members agreed to recommend validation of this approach. In Barton's empirical approach, shear strength is determined by the following equation:

$$\tau = \sigma_n \tan\left[JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_b \right]$$

where

- τ = shear strength of joint
- σ_n = normal stress across joint
- JRC = joint roughness coefficient varies from 0 to 20
- JCS = joint wall compressive strength
- ϕ_b = basic friction angle of joint wall
- ϕ_r = friction angle of weathered joints
- ϕ_b = ϕ_r for weathered joint

Recommendations

Process of Shear Strength Selection

27. Potential Mode of Failure. A detailed geologic mapping (showing joint orientations, dips, etc.,) of the foundation should precede obtaining test samples for determination of shear strength parameters. Based on geologic mapping, appropriate potential failure planes should be determined. For this purpose, approximate location and orientation of proposed structures (in the case of new structure) must be known. For existing structures, only the direction of loading needs to be known.

28. Representative Samples. Samples for testing should be obtained from the zones through which failure planes have the potential to develop. Appropriate boring techniques should be used to retrieve undisturbed samples of weak seams. Samples of 4-inch to 6-inch diameter are more appropriate than NX size cores for determining shear strength parameters. When exploration work is done by contract drillers, an experienced government geologist should be at the site to oversee the exploration work and to make decisions related to selection of test samples.

29. Parametric Studies. A sensitivity analysis by varying the values of shear strength parameters, c and ϕ or using Barton's empirical approach and varying the Joint Roughness Coefficient (JRC) should be performed before

embarking upon a detailed testing program. By performing a sensitivity analysis, one may be able to justify conservative shear strength parameters, and eliminate costly testing programs in many instances.

30. Testing Program Required and Evaluation of Test Results. Based upon the sensitivity studies, testing programs should be designed. If the sensitivity studies indicate that the basic angle of internal friction, ϕb, alone provides an adequate factor of safety, it is not necessary to conduct studies or testing for asperities of clean joints. In normal conditions, tests should be performed to determine the basic friction angle (ϕb), residual friction angle (ϕr), asperities angles (i) with respect to external loads, friction angle of gouge materials, etc.

31. Selection of Design Shear Strength Parameters. Selection of design shear strength is based on the failure criteria adopted. Consideration should be given to selecting a factor of safety based on the strength parameters used. Current industry practice considers peak strength as the failure criterion, and the shear strength parameters corresponding to the peak strength are used for stability analysis. For presheared material, however, the shear stress deformation curves do not exhibit peak stress and failure criteria should be determined by consideration of deformation. Since there is a complete lack of information in Corps manuals for determining failure criteria based on deformation, the necessary research is recommended to provide guidelines on failure criteria based on deformation.

32. Progressive Failure Effects. Shear failure along discontinuities within a rock mass is often accomplished through progressive failure, by which the maximum shear strength is not mobilized simultaneously along the entire failure surface to a residual shear strength. Gouge-filled joints should be investigated for such failure and the designers should be warned of this effect.

33. Summary of Recommended Research, Development and Standardization.

a. Develop a classification of joints based on strength parameters.

b. Conduct research for the determination of effective asperity angles, i, and their orientation.

c. Conduct basic research for obtaining ϕr for weathered joints.

d. Standardize testing procedures for determining shear strength parameters for the concrete-rock interface.

e. Standardize special tests.

f. Standardize the Direct Shear Test.

g. Standardize borehole index tests for thin soft seams.

h. Validate Barton's Empirical Approach to determine the shear strength of discontinuities.

i. Develop guidelines on failure criteria based on presheared materials and progressive failure conditions, or selection of a safety factor based on the shear strength parameters used.

FOUNDATION EXPLORATION PROCEDURES FOR STRUCTURE STABILITY EVALUATION

Chairman: Paul Hadala, WESGV-A (Geophysics, Civil Engineering)

Recorder: James B. Warriner, WESGR-M (Geophysics, Rock Mechanics)

Members: Neal H. Godwin, Jr., SWDCO-O (Civil Engineering)
Brian Greene, NCBED-DD (Geology)
Lawson E. Jackson, Jr., SWDED-G (Geology, Grouting)
Robert John, ORPED-G (Geology)
Todd Riddle, LMVED-GG (Geology)
Richard Wright, NANEN-DG (Civil Engineering)
Bob Yost, ORHED-GG (Geology)

Goals of the Working Group

1. The theme of the Working Group discussion was foundation exploration procedures for acquiring test samples and identifying weaknesses in the foundation for evaluating the sliding and overturning stability of existing concrete structures. The goals of the Working Group were to:

a. Summarize the present procedures, listing references.

b. Identify shortfalls in the present procedures.

c. Recommend potential solutions to overcome the identified shortfalls with emphasis on research and development plans to address the problems.

Introduction

2. The theme of this Working Group was chosen because of the acknowledged importance of actual geomechanical and geohydrological conditions beneath (and behind) existing structures in the realistic determination of structural stability and improvement thereof. Instabilities may be hypothesized and movements may be observed but, without actual examination of the rock-soil-water systems associated with the suspected instability, the structural behavior cannot be quantified, the degree of risk associated with structural behavior cannot be assessed, and the chosen rehabilitation techniques will not necessarily be appropriate or economical. The members of the Working Group were selected on the basis of their familiarity with geology as it affects engineered structures and their experience in the difficulties of performing subsurface exploration in intimate contact with major operating water-control structures. A typical set of circumstances has been observed by all participants. First of all, there is the question of the reality of the perceived instability: Is this decades-old structure in trouble or were the rules of the evaluation merely changed? If the former, then there is a subsurface condition which can be targeted by exploration strategy; if the latter, then the geologist engineer is exploring for a subtle or intangible end. Second come the artificial, but very real, problems of imposed policies and regulations: What are we allowed and what are we forbidden to do to locate the seat of instability? Will the work be done in-house or by contract? What contract

terms will obtain the data or samples we need? Why can't we dewater the lock since it is the only way to get a geologist on the floor? Next arise problems exemplified by the frequent necessity of getting core drilling apparatus into confined internal galleries. Finally there is the uncertainty, common to all geological exploration, as to sufficiency of effort. Though often controlled by available funds and time, the geologist-engineer can never be entirely confident that he has recovered enough samples or logs to fully characterize geotechnical phenomena. Five core borings or fifty, the designer's rehabilitation scheme will be of little benefit if a single critically located weak seam is missed by the pattern of holes.

3. To attempt to counter the above-stated complexities and barriers, the Working Group first discussed the goals set forth for its examination - their pertinence, the sequence of their importance, and how they relate to the subjects of other Working Groups. The methods and tools available for exploration were listed along with discussions of their relative values and appropriate applications. Procedures for planning and accomplishing explorations directed towards obtaining data for stability evaluations and rehabilitation designs were examined in general and by reference to specific projects within the experience of the participants. Shortfalls in those outlined procedures were specified as observed in specific past projects. Means by which the observed shortfalls have been overcome were described by the individuals involved and additional improvements in procedure were suggested. Areas of optimum value and future research and development applied to geoexploration within the REMR program were outlined. Finally, a statement reporting the Working Group's findings was prepared for presentation to all Workshop participants.

Assessment of Working Group Goals

4. The goals of the Working Group were examined for pertinence, priority, and relationship to other topics of the Workshop. Presently available geoexploration methods applied to existing structures are essentially extensions and adaptations of the methods used in the predesign, design, and construction phases of new projects. As such, they are described in a number of Engineering Manuals and Technical Letters. Table 1 is a partial listing of applicable manuals. Additionally, ASTM Standards, public technical literature, and private commercial vendors are all freely used within Corps FOA's in performing geotechnical exploration programs associated with existing structure evaluations. Thus, examination of geoexploration procedures was acknowledged as pertinent and having high priority. However, given the ready access of engineers to voluminous references and numerous specialty consultant services, few shortfalls were identified in the application of current methods of geoexploration.

5. The Working Group felt that we have the tools with which to explore and examine the area below and around existing structures. What we do not have in-house, we can hire. But, still there are difficulties when we try to get information to perform stability analyses and plan repairs or rehabilitation programs. Shortfalls in the actual accomplishment of geotechnical investigations for stability evaluation were identified as principally nontechnical in origin or nature. The consensus was that problems generally fell within the areas of administration, communications, and regulatory complications.

"Shortfalls," then, do exist, though not necessarily of a technical nature. They are of notable importance and are amenable to solution.

General Aspects of Geotechnical Exploration

6. In any project evaluation, the problem must be defined and then the exploration program planned around that definition. Immediately, the Working Group observed, a difficulty arises in ascertaining the reality of a perceived problem. If there is a degree of reality to perceived structural problems, then there will be targets at which the exploration program can be aimed. If, however, there are no observed evidences of structural distress or if the structure has never even been loaded to analysis case conditions in which the analysis results predict failure, then the exploration program must be directed toward diffuse and subtle targets ("ghosts" in the words of the Group). In the former, a specially directed exploration program can be used (i.e., core samples of weak seams). In the latter, a more regional exploration program is required (i.e., the core borings are more numerous and placed in idealized patterns). In the collective experience of the Group, the latter, broad-based exploration often produces broad spans of results - "fuzzy" characterizations ambiguous both in location and parametric values. In the case of the generalized exploration, the engineer/geologist directing the exploration usually feels called on to use experience and judgement, coupled with early results, to define the critical zones or phenomena which place the structure's stability in question.

7. Whatever is the nature of the supposed critical zone or phenomenon, the foremost directive to both the exploration manager and the structural analyst is to KNOW THE SITE GEOLOGY. Reports of prior investigations, whether from Construction Reports, Periodic Inspection Reports or from nonspecific geologic studies, are the single most valuable types of guidance in planning exploration programs. Experience and ingenuity assist in locating documented background information. On-site reconnaissance is valuable in giving definition to geoexploration targets. It was pointed out that geologists and engineers look for and describe things pertinent to structures differently than was done in the past. Evidences of preexisting mass movements are now actively searched for and related to both the structure and any impounded reservoir or channel; and the size scale of "believable" mass movements is much larger than in the years prior to Libby and Vaiont Dams. Rock joints and shear zones are now taken as real and critical facets of geology in terms of both mechanical behavior and hydraulic properties (e.g., Malpasset and Walt Creek Dams).

8. The role of geophysics was observed to be almost as uncertain as the confidence placed in its results. Some Districts and Divisions have essentially no continuing capability for any geophysical investigations, either in active use or in judging their applicability or the value of their results. Other offices actively direct in-house equipment and specialist personnel toward all phases of exploration and still others remain cognizant of the state of the art and freely engage contracted geophysical services for complex problems involving existing structures. The Lower Mississippi Valley Division (LMVD) routinely uses electric logs as a strata correlative tool, even to the point of placing borings solely for logging purposes. Caliper logs and natural gamma radiation logs, coupled with the electric logs, have been applied by the Division for weak seam determination. The North Central Division (NCD) and South Atlantic Division (SAD) have hired geoelectrical

streaming potential surveys for water seepage path location and subsurface radar surveys for location of anomalous voids and solid bodies. The NPD has contracted for acoustic logging and cross-hole seismic surveys to determine the elastic properties of rock foundations and grouted rock masses. Corps-wide, there is no real uniformity to reliance on geophysically derived data, any more than there is on the application of geophysical techniques. In some project cases geophysics is used as an early guide to directing later, more detailed core sampling exploration. In some cases geophysics is used midway in a program to extend knowledge gained from early boreholes to uncored areas.

9. Core borings and sample recovery methods were described as being, in practice, virtually standardized if accomplished by in-house drillers. The expertise and equipment exists, it was felt, within the Corps to make holes and recover samples from practically any project site. Problems and shortfalls do, however, arise at nearly every project in the form of obstacles to management and planning. There is a recurrent trade-off between funds and manpower available on one side and the number and depth and sizes of borings believed required to adequately characterize the geologic/hydrologic conditions. Always present, this difficulty in optimizing resources available versus effort required has become greater and more restrictive in recent years. There are practical problem areas in the execution of drilling programs in existing structures, such as drilling from within the confines of internal galleries or conducting drilling during structure operational use, but lack of money and knowledgeable personnel were seen as the greatest obstacles to in-house drilling.

10. Contracted drilling and sample recovery is a means of bypassing personnel constraints, but the actual practice leaves much to be desired in the opinions of the participants. Assurances on the quality of recovered samples are difficult to obtain, especially if the contract payment involves total feet or numbers of holes drilled. There is seldom practical recourse for the geologist/engineer in charge of a contracted drilling program to enforce any kind of quality control. This condition is made more severe by the fact that the weakest characteristics and zones of rock are those most likely to be critical to structural stability but are also the most difficult to sample. It is impossible to distinguish between real, critical zones of weakness and sloppy sampling techniques when the decision is based solely on fragmented or lost core.

Application of Exploration Effort

11. In some circumstances the number of holes drilled or samples recovered is based upon the scatter and the values of laboratory test data from previous studies. The geologist/engineer in charge of exploration then becomes a participant in the analysis and design process by way of his direct experiences in the pattern of recovered samples and the degree of variability of the targeted strata. His knowledge of the geology gives the person in charge of sampling a unique insight into the relative importance of specific test results and the data spread. For reasons arising both in the offices of the structural analysts and at the field offices, that direct knowledge and experienced insight frequently either does not get passed on or is boiled down to a single average or extreme value for a parameter.

12. Complementary to the above point made in the Working Group discussions was the statement that "the best people for a particular job are usually back in the office away from the project and often tied up with another job." Whether justified or not, the fact observed is that the most experienced and qualified people for interpreting geological and engineering data and solving problems are too occupied to provide substantial input to on-site plans and decisions.

13. In situ tests were acknowledged to always be better than laboratory tests, if properly carried out. They provide elastic parameters, strengths, and pore pressures. Shear strength tests were stated to be the type on which the most time and money were spent. But those test specimens are of such small size that only small-scale roughness features are included in the test results. The realistic scale of features to be tested should be much larger (and more expensive) and, with that knowledge, the bulk of shear test results are ignored in favor of the lowest, or nearly the lowest, value found. This procedure amounts to little more than an educated assumption. It is incongruous, therefore, that numerical values for strength, etc., are often demanded by structural analysts to a level of precision not justified by either the sources of the data or their application.

Specific Shortfalls and Recommendations

14. Location and characterization of asperities or roughness features on rock interfaces was stated to be a shortfall in the current exploration practice. This problem correlates exactly with one pertinent to determination and use of shear strength; namely, what part does realistic discontinuity roughness play in shear strength and sliding stability? No conclusions were reached other than that of the importance of the subject.

15. Anticipation of the potential modes of failure of a structure on rock was accepted to be the responsibility of the exploration team. To cope with that responsibility it is necessary to devote personnel to specific projects who have a wide-based knowledge of prior case histories and personal experiences. Specialized training courses are needed to increase project personnel familiarity with how rocks and rock masses can behave and, in so doing, how they can threaten existing structures.

16. Never rely on a single method of exploration, whether it be drill core or a specific geophysical survey. Rather, when possible, plan a "suite" of core borings, in situ tests, and geophysical surveys, each of which complements and supplements the type of data obtained by the others.

17. The question was posed as to the degrading effects on rock materials of free and impounded water and imposed vibrations. Though the participants were aware of that question being raised in past years, none had any observations of conditions that could be interpreted as long-term material degradation.

18. Accessibility of drilling and survey equipment into unusual and cramped portions of existing structures is a frequent problem. Communications in the form of pictures or other notes of past experiences which overcame this problem could be distributed in a Technical Note.

19. Problems arising from experienced Corps personnel being unavailable for project-specific problems were seen to be surmountable if the need for help was expressed sufficiently. What is lacking is a form of directory (other than in people's minds) as to what expertise does exist in the Corps nationwide and where those individuals are on duty.

20. A number of suggestions were offered for dealing with "unmotivated drilling contractors." They include payment by time spent rather than feet drilled; payment for acceptable samples recovered rather than feet drilled; assignment of experienced geotechnical personnel as Contracting Officer's Representative rather than someone in a clerical position; and specifying in the contract terms exactly how many and by what means samples are to be recovered and preserved. A Corps-wide survey of drilling contract language, both good and bad, was seen as necessary and beneficial.

21. Rock sample handling was discussed to ascertain possible desirable research. Knowledge of sample handling methods was found to be as varied among the participants as were the actual practices used in their organizations. An updated survey and evaluation of sample preservation and transportation methods that have come about since the 1930's was felt to be worthwhile. Softer (e.g. Tertiary age sedimentary) rocks and discontinuity sample handling requires original research and development. Semi-automated, thermally sealed plastic wrapping was suggested as an area of study.

22. In order to bypass the problems inherent in sampling rough discontinuities of large scale it was suggested that the discontinuities be characterized in situ. A variation on cross-hole electromagnetic wave surveying was suggested in which the discontinuity was "doped" with injected metallic salt solutions, thus providing a target for the electromagnetic wave survey. The proposal was acknowledged to have a high risk of failure but to offer the potential ability to "see" discontinuities in their totality between boreholes.

23. Geophysical methods were discussed at length, beginning with electric and natural gamma logs. If there is good prior information on the geology, then we can definitely use the geophysical logs with confidence early in the exploration program to correlate strata and identify clay-rich (often weak) zones. If there are no or poor records existing on the site geology, then the logs should be used in every boring, but only minimal reliance should be placed on log interpretation in the earliest borings. Experience and geologic insight must be developed at each site before the logs can be used with confidence. Standard seismic and electrical geophysical surveys play little part in structural evaluation; they are simply not directed to the desired features of rock masses. Cross-hole seismic surveys, however, can provide elastic parameter data if they are required for stability analysis.

24. Remote sensing of geological features appears to function on too large a scale to be applicable to existing structure reevaluations; but developments by other agencies may arise.

25. Thermal surveys, whether of water bodies or ground water or regional infrared radiation patterns, offer some promise in the future but not yet.

26. A question arose during discussions as to using geophysical methods, specifically acoustic ranging similar to sonar and analogous to radar, to find and characterize the concrete-to-rock interface under existing structures. Some participants reported observing voids at that interface, some reported little or no adhesion between rock and concrete, and all agreed the configuration as-built never matched the design/construction records. Such an acoustic method was agreed to offer potential benefits arising from its development. The implications of possible voids or nonintimate contacts to both sliding stability and to uplift estimation were discussed.

27. A major and strongly desired action on the part of the REMR program was the establishment or adaptation of a data base of rock strength test results. The data base would be similar to or part of the Computer-Aided Geotechnical Engineering (CAGE) system and be accessible nationwide. The goal would be for a local engineer/geologist to be able to extract all test results for particular lithologies or specific formations or from single localities that had been obtained in prior test programs. It was stated that even within the confines of some Districts, there was no single source of prior rock characterization information.

Table 1

GEOTECHNICAL INVESTIGATION GUIDANCE SOURCES

EM 1110-1-1804	Geotechnical Investigation for Civil Works and Military Construction
EM 1110-2-1908	Instrumentation of Earth and Rock Fill Dams
Part 1	Ground-Water and Pore Pressure Observations
Part 2	Earth-Movement and Pressure Measuring Devices
EM 1110-2-1803	Subsurface Investigations - Soils
EM 1110-2-1907	Soil Sampling
EM 1110-1-1802	Geophysical Exploration
EM 1110-2-1906	Laboratory Soils Testing
	Rock Testing Handbook

COMPUTATION OF FORCES AND METHODS OF ANALYSIS FOR STRUCTURAL STABILITY EVALUATIONS

Chairman: Don Dressler, DAEN-ECE-D
Recorder: Carl Pace, WESSC

Members : Erville Staab, MRD
Hubert Deal, TVA
Don Logsdeon, NCR
John White, SPK
C. W. Kling, SAM
N. Radhakrishnan, WESKV
Don Chambers, NPP
John Peters, WESGE
Joseph Erhart, NCB
Jerry Foster, FERC
Howard Boggs, USBR
Wayne Clough, Virginia Polytechnic Institute
Fred Tracy, WESKA-E
Dale Munger, DAEN-ECE-G

Goals of the Working Group

1. Summarize the problems and shortfalls associated with determining the driving and resisting forces on a concrete structure on a rock foundation.

2. Identify the shortfalls in the present procedures which are used in evaluating the stability of existing concrete structures on rock foundations.

3. Recommend potential solutions with emphasis on research and development activities to overcome the identified shortfalls. Field and model testing as well as analytical analyses should be considered.

Introduction

4. The majority of the Corps of Engineers (CE) lock and dam structures are 30 to 100 years old and have shown no signs of instability. However, many of these structures do not meet present-day stability criteria when analyzed by conventional stability analysis methods. Some of these structures even have monoliths which have a calculated safety factor of less than one. Since these monoliths have not failed, it is obvious that the calculated safety factor is incorrect. In addition, overturning analysis of the monoliths of some structures yields results which indicate that the monoliths will overturn when subjected to loads of a magnitude that they routinely withstand. Calculating errors may be due to inadequate determination of applied or resisting forces, inadequate selection of analysis parameters, and/or the concepts used in the stability analysis and evaluations being invalid.

5. The conventional stability analysis of an existing structure involves making a number of assumptions with respect to forces, analysis parameters, stability analysis methods, and evaluation criteria. These assumptions need to be studied and evaluated to make sure that we are not overly conservative

in our assumptions and that our stability evaluation results are truly representative of the in-place stability of existing concrete structures on rock.

6. The cost of strengthening a structure against sliding or overturning is substantial. This expense, however, should not be a reason to change our conventional stability analysis procedures unless it can logically be shown that the change is a better representation of the true in-place stability of a structure. The general feelings of the Working Group were that our stability procedures result in overconservatism and that we often spend dollars to strengthen structures which have adequate stability.

Force Estimation

Introduction

7. There are both applied and resistive forces which act on lock and dam structures. Examples of applied forces are soil and water loads. Examples of resistive forces are friction, cohesion, and strut forces which develop to resist the applied loads.

8. It is not practical to measure the total forces acting on a structure; therefore, they are estimated. The present way of determining these estimates for many of the applied and resistive forces on existing structures is crude because of restrictions and variables such as:

- a. Limited sampling and analysis of backfill and backfill pressures and limited measurement of uplift pressures due to limited funds.
- b. Inadequate methods for determining the existing backfill pressures on lock structures.
- c. Lack of instrumented data from existing locks and dams.
- d. Nonhomogeneous backfills.
- e. Inadequate information on the construction of existing lock and dams. For example, the as-built geometry may be in question, and the backfill material and its compaction may not have been documented.
- f. As-built drawings documenting the location of the concrete-rock interface are usually not available or are not accurate.
- g. Cores of the concrete structure and the rock foundation used for determining material properties, weak seams, location of the concrete-rock interface, disparities, etc., are expensive to obtain and at best give limited results about the three-dimensional environment.

Backfill Pressures

9. The need for improved methods for determining the backfill pressures on existing structures was discussed by the Working Group. A variety of

backfill materials have been used at CE structures, and many are nonhomogeneous, which further complicates the problem. Cobble backfills are extremely difficult to sample and test and backfill pressures are largely unknown. The determination of actual backfill pressure distribution on a structure is a problem the Working Group felt should be addressed. Research should be conducted on the effects of the shape of the landside lock wall monoliths on backfill pressures.

10. Devices which have been used to obtain soil pressures are: pressuremeters, Marchetti dilatometer, Gloetzl cell, Camkometer, and the lateral-stress cone. The success of these devices is dependent on the type of soil and the experience and knowledge of the operator concerning the device and its limitations. Additional research is needed on many of these devices in various types of backfills.

11. The determination of backfill pressures was discussed at the workshop by G. W. Clough, and his paper entitled, "Comments on A Proposed Investigation of Lateral Earth Pressures Exerted by Backfills" included in this report.

Uplift

12. There are differences in the CE and the USBR approaches to account for uplift on the base of lock and dam structures. The CE accounts for uplift as a load, while the USBR accounts for uplift as a base reaction (pressure). The USBR method of accounting for uplift is more conservative and may be more applicable to high-head dams, while the CE method may be more applicable to low-head dams. The Working Group concluded that much uplift data has been collected on various concrete hydraulic structures and that this data could be collected, put into a data base, analyzed, and the results correlated with the various assumptions used in analysis to draw conclusions about the validity of the assumptions.

13. There was some discussion on the effects of changing hydraulic head on uplift pressures and drainage system efficiency. It was felt that available piezometer instrumentation is sufficient to make the necessary measurements to determine the effect of changing heads, if the piezometers are installed properly and in the right locations. There may even be enough available uplift data, once collected and analyzed, to draw some meaningful conclusions on the effects of changing hydraulic head on uplift. Analytical studies should be performed to estimate drain effectiveness using well theory and the principles of fluid mechanics.

14. It was pointed out that there is also a difference in the way the CE and USBR account for uplift within the concrete sections of a hydraulic structure. The USBR assumes a hydraulic gradient which starts at the upstream hydraulic head on the structure and travels through the structure in a straight line to the elevation of tailwater. The CE assumes a hydraulic gradient which starts at an elevation of one-half of the upstream hydraulic head on the structure and travels through the structure in a straight line to the elevation of tailwater. This was not considered to be an item worthy of additional research as uplift is not generally considered to be a problem within the concrete structure.

15. Analytical studies by finite element analysis can be performed to estimate the tilting and crack development at a concrete structure-rock foundation interface of a structure suspected to have a crack at the interface. This analysis could then be compared to the results obtained by conventional analysis of the same structure. If the conventional or finite element analysis indicates that a crack could exist, then a borehole micrometer could be installed and measurements taken under changing loads to see if the crack really exists at the structure-foundation interface. A crack at the structure-foundation interface could change the uplift on the base of the structure.

Drains

16. Drainage systems can be used to reduce uplift pressure on the base of a hydraulic structure. These systems can lose efficiency with time, however, due to clogging, and some method is needed to determine drainage system efficiency and to determine when the system needs to be rehabilitated. There is the additional need to know how drainage efficiency is affected by changing hydraulic head (discussed previously in paragraph 13).

17. Current CE policy is to treat the structure as if it does not have a drainage system when a crack is estimated to exist at the structure-foundation interface and extend beyond the drain. Full hydraulic head is assumed to act on the base for the length of the crack. There was some discussion on whether this assumption was reasonable, and some members of the Working Group felt that the drainage system would still reduce the uplift pressure and should be considered in the stability analysis. It was felt that additional research should be conducted to resolve this issue.

Stability Analysis

Sliding

18. The CE uses the Limit Equilibrium Method as outlined in ETL 1110-2-256 for the sliding analysis of concrete structures on rock. One concern mentioned about this method was that it neglects the effect of strain compatibility. Many of the Working Group members considered this to be a serious deficiency when evaluating projects with soil backfill, a rock foundation, and a passive resistance system.

19. The various types of resistance (friction, cohesion, and passive) which cause a structure to be stable do not develop at the same rate in relation to the resultant applied load. An example of how comparative resistance rates may develop is illustrated in Figure 1.

20. The maximum magnitude of each of the various types of resistance can be computed in the conventional manner, but their developments, in relation to each other, may vary in phase. This is important because if the maximum of each resistance does not develop at the same strain or resultant applied load, it will never be possible to have a total resistance equal to the sum of their maximums. At the resultant applied load of H_1 in Figure 1, the total resistance would not be the sum of the maximums, but would be the sum of the specific resistances at H_1. This could cause a significant effect on the sliding safety factor as the applied loads increase.

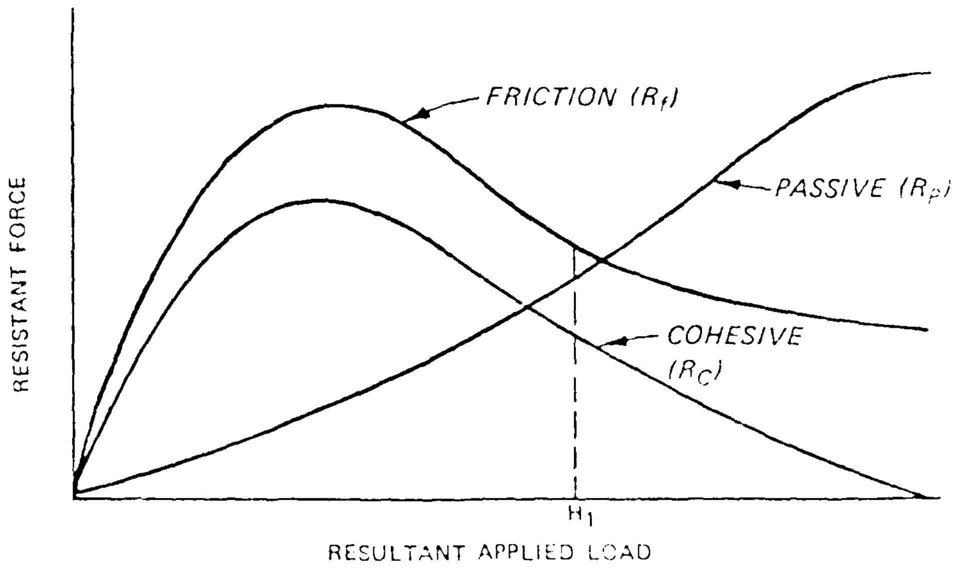

Figure 1. An assumed phase development of resistance

21. The USBR uses the Shear Friction Method for the sliding analysis of its structures, while the Tennessee Valley Authority (TVA) uses the Shear Friction and the Limit Equilibrium Methods. There were proponents of both methods at the Workshop. It was unresolved as to whether additional research should be funded to develop an improved analytical method for analyzing sliding stability or exactly what research should be undertaken. It was recognized that the two existing methods (Shear Friction and Limit Equilibrium) both have their deficiencies.

Overturning

22. For many years, the adequacy of a structure in overturning stability has been evaluated based on where the resultant of applied loads intersects the base of the structure. If the resultant falls within the middle third of the base, the total base will be in compression, and the structure is safe against overturning. For certain loading conditions, the resultant can fall outside the middle third and the structure can still be judged as adequate. For example, when "at-rest" earth pressures are used in normal operation, extreme maintenance, or maximum flood loading cases, the resultant of applied loads can fall outside the kern, but at least 75 percent of the base must be in compression. For operating conditions with earthquake, the resultant has only to fall within the base, but the allowable foundation stresses should not be exceeded.

23. Sliding analysis uses either developed forces (Limit Equilibrium Method) or maximum forces (Shear Friction Method) in determining the sliding safety factor. Problems can occur, however, if these same forces are used in the analysis of overturning because some of the passive resistance forces considered for sliding analysis may not be capable of fully developing in an overturning situation. Further, the forces obtained from the Limit Equilibrium Method are based on assumptions and are not the actual forces on the structure. The sliding and overturning modes of behavior are coupled in nature, and it would be best to analyze both modes using consistent forces.

ANALYSIS OF EXISTING STRUCTURES VERSUS THE DESIGN OF NEW STRUCTURES

Introduction

24. The information available for the evaluation of existing lock and dam structures is significantly different from that available for the design of new structures. The existing structure has an operating history of structural response to various case loadings which in most cases gives clear implications about the safety of the structure. Some would say that, if a structure has operated without any signs of instability for 30 to 100 years and has been subjected to various loading conditions for which it was designed, then the structure is stable and does not require strengthening.

25. For an existing structure, the overall structural performance during various loading conditions will be known; borings and written comments about the as-built conditions of the foundation may be available; construction techniques may be known; and the existence of the structure will allow actual material parameters for the structure to be obtained. However, this information is usually limited and stops short of what is needed, but it is still helpful. It is more difficult and expensive to collect data on the foundation of an existing structure, due to limited access, than to collect data on the site of a proposed structure.

26. The differences in the analysis of existing structures versus the design of new ones prompts a consideration of adopting a different or new attitude about the stability criteria of existing structures, the monitoring of the response of the existing structure, and their maintenance.

27. The analysis of existing structures should use practical concepts. For example, a conservative analysis can be used for an existing structure, and, if it meets present-day criteria, the expense of additional investigations necessary for a less conservative analysis is not needed.

General Comments

28. The determination of applied and resisting forces and the concepts used in the methods of analysis and evaluation of the stability of existing concrete structures or rock foundations should be considered from an overall viewpoint. The whole problem must be considered when making changes to existing methods of analysis because many interrelated factors contribute to the overall safety of the structure. Care has to be exercised when changing one factor in the analysis because the structure is safe due to many factors and a consistency of conservatism must be maintained without causing an accumulation of safety factors which causes the analysis to be excessively conservative.

29. Many problems encountered in stability analysis are site-dependent, and care should be taken to delineate site-dependent effects and not change the criteria as applied to all structures because of an isolated problem. The determination to deviate from standard procedures for: (a) obtaining forces, (b) selecting analysis procedures, and (c) determining evaluating concepts should be based on a good exploration and instrumentation program which gives

the actual pressures, forces, and deflections at or near the structure as well as material properties and relationships.

30. An existing structure has the advantage of having an operating history. If the operating history covers a reasonable range of loading conditions and the structure has shown no signs of instability, consideration should be given to factors not considered in the design of new structures but which contribute strength to the stability of existing structures.

31. The existing locks and dams which do not meet present-day stability criteria are numerous, and in almost all cases their operating histories show no stability problems. This is not conclusive evidence that the structures should not be strengthened in stability. However, it does strongly indicate that many of these locks and dams are stable and that overconservatism exists in the loadings, stability parameters, stability analysis, or stability criteria.

32. The friction angle (Ø) and cohesion (c) values used in stability analysis should be obtained as realistically as possible. The upper and lower bound values for (Ø) and (c) should not be used unless they are used in parameter studies to guide the stability evaluation.

33. Comparative studies should always be made on any aspect of the stability analysis and on methods available for strengthening structures in stability when existing structures are analyzed and strengthening measures considered. Computer programs developed by the CASE committee should be helpful in performing efficient stability analysis.

34. In cases where structural geometry or loading is unsymmetrical, a three-dimensional (3-D) stability analysis may be necessary to obtain reliable results.

Recommendations

35. The feasibility of measuring backfill pressures in various types of backfills should be studied and an assessment made of available testing techniques. Test programs for consideration are:

a. A limited test program using a high-pressure pressuremeter to measure the backfill pressure for a cobble backfill. Consideration should be given to how close measurements should be made from the structure-backfill interface. So that comparisons and evaluations can be made, the measurements should be performed where other instrumented pressure values have been obtained.

b. A similar test program could be conducted using a pressuremeter in a clay backfill to determine the variation of backfill pressures with distance from the backfill-wall interface. These results should be compared with stress cell data.

c. Arrays of Gloetzl cells should be placed in the backfill behind a structure to measure the changes in backfill pressures for a range of loading conditions. An instrumentation system should be used which is adequate to monitor these measurements over a long period.

36. It is recognized that in order for a stepped or irregular shape structure covered with backfill material to overturn, a shear plane must develop through the backfill material. Resistive shear forces develop along this shear plane during structure tilting which resist overturning, but they are not considered in stability analysis. For some structures, this could mean that a significant resistive force is being ignored in the overturning analysis. As assessment should be made of the influence of this force on overturning stability.

37. A parametric finite element study should be performed to determine the loads on a structure with a soil backfill, rock foundation, and passive rock or soil resistance, and the results should be compared with the loads obtained from the limit equilibrium analysis as presented in ETL 1110-2-256. Strain compatibility should be considered in relation to both analyses to improve the understanding of the behavior of the structure and backfill and what, if any, changes should be made in the present stability analysis.

38. Existing data should be collected from various organizations (TVA, USBR, CE, etc.) on uplift measurements and other parameters needed in the analysis of the uplift under the structures. These data should be loaded to a data base, analyzed, and the results correlated with uplift pressures which were or would have been obtained by the usual design assumptions. Determinations should be made as to the validity of the CE and USBR approaches to account for uplift on the base of lock and dam structures. Recommendations should be made concerning how uplift should be used in stability analysis.

39. A realistic model should be developed of a monolith of a lock or dam, and a finite element analysis performed to obtain the tilt and base pressures. A conventional stability analysis should be performed for the monolith and the results compared with those from the finite element analysis. It should also be determined if conventional analysis gives a true representation of the likelihood of a crack existing at the upstream or loaded face of the structure-foundation interface.

40. Experimental tests should be performed on a block structure, and crack development should be measured at the structure-foundation interface under loadings. Finite element and conventional stability analyses should be performed for the test structure under the same loadings, and the calculated cracking at the structure-foundation interface should be compared with the experimental test results.

41. Experimental tests should be conducted to determine if it is feasible to allow some tension due to bond at the structure-foundation interface. TVA allows 15 psi of tension at the structure-foundation interface.

42. An existing lock or dam monolith should be instrumented during rehabilitation to determine if a crack really opens at the structure-foundation interface under loading when conventional stability analysis indicates there should be a crack.

43. Drainage system efficiency should be evaluated under various conditions, and a theoretical model based on well theory and fluid mechanics should be developed or validated. Rehabilitation techniques for drainage systems should also be studied.

44. A study should be conducted to determine whether or not the sliding safety factor should be constant or variable in relation to the various applied and resisting forces. Strain compatibility should be considered in this study. A realistic sliding analysis procedure should be developed using variable safety factors for various applied and resisting forces.

45. A rational approach should be developed for the analysis of existing structures which takes into account the additional information which is known about the operating history, the site, and the loadings on the existing structure. This approach should provide uniform guidance on the forces to be used in sliding and overturning analysis.

46. Laboratory tests should be conducted to determine how much movement can be tolerated for a concrete structure on a rock foundation without significantly decreasing its resistance in stability. This may have an impact on the use of stressed or unstressed anchors because the unstressed anchors must have movement to develop resistive forces.

Summary

47. Factors which may contribute to existing lock and dam structures not meeting present-day stability requirements are:

a. Inadequate consideration in the analysis of all the information which is available for existing structures including their loading history.

b. Inadequate determination of applied and resisting forces.

c. Inadequate selection of analysis parameters.

d. Use of invalid concepts in the stability analysis and evaluation.

48. These factors should be studied and recommendations made for changes in the overall stability evaluation which will better define the in-place stability of the existing structure.

INSTRUMENTATION AND MONITORING PROCEDURES FOR THE PURPOSE OF EVALUATING THE STABILITY OF EXISTING CONCRETE STRUCTURES ON ROCK

CHAIRMAN: Stephen Deloach, USAE-ETL
RECORDER: Avinash C. Singhal, WESSC-R/ASU

MEMBERS: Robert L. Anderson, WESGR-M
Howard Boggs, Bureau of Reclamation
Harold C. Buttrey, TVA
Lewis A. Gustafson, NPDEN-GS
Brian Kleber, LMSED-FI
Kalman Kovari, ETH-Zurich
Edward Loftin, SWLED-DS
Robert Snipes, SAWEN-GS

Goals of the Working Group

1. Summarize any present procedures and experiences for designing, implementing, recording, and evaluating instrumentation for the purpose of evaluating the stability of existing concrete structures on rock. Include both continuous monitoring instrumentation and specially designed instrumentation for identifying the level of safety of existing concrete structures.

2. Identify the shortfalls in the present procedures and address what types of monitoring of existing concrete structures can best aid in the stability evaluation problem.

3. Recommend potential solutions to overcome the identified shortfalls, with emphasis on R&D plans for addressing the problems and needs.

Introduction

4. The REMR Workshop on "Assessing the Stability of Concrete Structures on Rock" brought forth an exchange of ideas from leading experts in the area of "Instrumentation and Monitoring Procedures." Each Workshop group participant was asked to provide a two-page essay describing his views as to what topics should be addressed by REMR. In order to present a simple but meaningful report on the workshop proceedings, the essays of the participants have been summarized in Table 1. From this tabulation, a consensus was established in relation to three main topics. They are Instrumentation, Data Collection, and Criteria and Standards, and each is discussed in the body of this report.

5. The Instrumentation Working Group attempted to focus upon the following topics:

a. Summarizing present procedures and experience for designing, implementing, recording, and evaluating instrumentation for the purpose of evaluating the stability of existing concrete structures.

b. Identifying the shortfalls in the present procedures and specifying the different types of monitoring systems which best aid in the stability evaluations.

c. Recommend potential solutions to overcome various shortfalls with emphasis on research and development plans.

Instrumentation

Types of Measurements

6. For dams and their appurtenant structures, the main quantities measured are: (a) loads, including both forces and pressures, (b) displacements or movements, including both absolute and relative values, (c) strains which are usually converted into stress values, (d) temperatures, (e) seepage flow rates, (f) vibrations (i.e., accelerations), including seismic motions, (g) acoustical emissions, and (h) rotation.

Types of Instruments

7. Measurements are made by mechanical, electrical, optical, and acoustical methods. Some of the commonly used instruments are: (a) force gages, (b) pressure gages, (c) extensometers, (d) tiltmeters, (e) inclinometers, (f) plumb lines, (g) surveying (including electronic distance-measuring devices), (h) strain gages, (i) stress gages, (j) linear variable differential transformers, (k) crack gages, (l) joint meters, (m) thermometers, (n) thermocouples, (o) flowmeters, (p) piezometers, and (q) seismographs.

Instrument Installation

8. After World War II, instrumentation became a part of dam design. Early in this period, relatively little instrumentation was included. As the need for more information became apparent, additional instrumentation was added. Adding instrumentation to an existing structure is more expensive than if it is installed during the original construction and often does not provide all the desired information or is lacking in sufficient accuracy.

Design Group Visibility

9. Over the years, instrumentation design has become more sophisticated. Present practice is to include instrumentation as a part of the design, but several Workshop participants indicated that it is still relegated to a secondary role in the design process. Instrumentation should be considered an integral part of the design process and not added as an afterthought.

Reliability-Accuracy

10. One area of apparent deficiency in present-day instrumentation is that construction/first loading and long-term safety and stability measurements are obtained from the same instruments. These two functions should be separated, and the safety measurements should be taken from a set of instruments dedicated solely to this purpose. On most of the concrete structures designed or operated by the Corps of Engineers, there are two kinds of instruments,

namely: (a) standard or common instruments such as plump lines and piezometers, and (b) special or investigative instruments. Often, there is an overlap in these two kinds of instrument systems. The investigative instrument package depends upon the nature of the problem under investigation; for example: *deflection versus cracking in the rock mass versus leakage.* The Office, Chief Engineers, US Army currently has design guidelines (ER 1110-2-4300) and recommendations for various instruments which must be installed in all new concrete structures.

Instrument Availability

11. Many new instruments are now available as a result of new research and developmental efforts by various manufacturers. Keeping abreast of new instrumentation, including their availability and capabilities requires considerable effort on the part of any instrumentation design group. It is recommended that a catalogue, perhaps in the form of REMR notes, be produced which identifies the availability, capability, accuracy, and expected life of presently available instrumentation. This catalogue would also serve to indicate areas where instrumentation is lacking or has not been developed or which instruments currently have limited performance.

Data Collection

Manual versus Automatic

12. Data collection, when done manually, is subject to operator errors and/or a lack of skill in reading the instruments. This situation is especially true for dam and lock operation, because technicians are often not informed or lack training in the significance of various data. An automatic data collection program would do much to overcome many of the problems resulting from the manual collection of data.

Data Analysis

13. Once the data are collected, most Workshop members agreed that the analysis of the data is quite well handled by the various agencies. At most installations, design personnel who were often members of the original design teams are available for the data analysis. These personnel have the skill necessary to find any anomalies or inconsistencies in the data. Whenever deviations occur, the agency is then prepared to take corrective action as needed.

14. The TVA experience of a single agency designing, constructing, and operating a dam permits a continuity of personnel and ideas, which is clearly advantageous over the situation of disjointed groups performing these functions separately. TVA and the US Army Corps of Engineers emphasis periodic training for all their field personnel who collect data. Data are plotted and documented on a continuing basis to look for adverse trends. A statistical package should be developed which would automatically monitor the output of the safety measuring instruments at each concrete structure site and indicate potential problems well in advance of any crisis situation.

15. Similarly, Bureau of Reclamation engineers design the concrete dam and the instrumentation package and monitor the structural behavior continuously

through construction/first filling and well into operation. Thus, the design criteria and other parameters may be effectively evaluated.

Criteria and Standards

Consistent Criteria

16. A nearly unanimous agreement was expressed by the Workshop members in the need for cooperation and interaction between the existing agencies in order to produce a government-wide set of standards or guidelines. The purpose should be to replace the existing agency standards, which are in some instances contradictory. The design of an instrumentation package for each individual dam is dependent upon the expertise of the design team and the importance placed on instrumentation. For some projects, the instrumentation is well planned and integrally constructed, while for other projects the instrumentation takes on a secondary importance. A government-wide guideline would be of considerable help in this area.

17. Presently, the interpretation of both operating and safety data is done against the existing criteria. The standards for different agencies are in some instances contradictory. Some of the standards have been modified since the original design calculations. Therefore, there are a significant number of dams which do not meet present safety standards, primarily for the uplift-force criterion. Some of these dams have been reengineered and modified to bring them into safety compliance, while others have not.

Safety Criteria

18. Several of the Workshop members felt that the various agencies probably now have enough historical data on uplift-forces so that a design standard could be developed. If this were accomplished, they also expressed optimism that many of the dams which do not meet existing stability standards in this area could meet the new, more comprehensive standards. For the few remaining dams which would not meet the new standards, a significant reengineering program could be undertaken in order to bring them into strict stability compliance.

19. Most of the lack of stability compliance in concrete structures today results from the uplift force calculations. However, another significant area of safety concern, where there is little information available, is in the area of geological slippage in shale and clay seams.

Shortfalls

20. It is recommended that the field instrumentation data be compared with the basic parameters and assumptions used in the design stages of the project. The data should be used to verify whether the structure behaves as designed. One serious shortfall is the lack of reliable instruments that can accurately measure existing stresses within soil, rock, and concrete. Methods used currently provide variable results and are somewhat unreliable.

Conclusions and Recommendations

21. Many different types of instruments are currently used for concrete structures, namely: (a) force gages, (b) pressure gages, (c) extensometers, (d) tiltmeters, (e) inclinometers, (f) plumb lines, (g) survey instruments, (h) strain gages, (i) stress gages, (j) linear variable differential transformers, (k) crack gages, (l) joint meters, (m) thermometers, (n) thermocouples, (o) flowmeters, (p) piezometers, (q) seismographs.

22. Most of these measurements are made by mechanical, electrical, optical, and acoustical methods.

23. It is best to properly design an instrumentation package and install it during the original construction. Instrumentation design groups should be elevated in stature and visibility to a level equal to other design groups. Many instruments cannot be retrofitted (e.g., plumb lines) into existing structures.

24. Instruments are often called upon to work in a "hostile environment" for almost the entire life of the concrete structures. Additional research is needed on the reliability, maintainability and accuracy of these instruments. Installation of instrumentation is a key area. Improper installation can negate perfectly good instruments.

25. It is recommended that REMR notes be produced to identify the availability, and capability of currently existing instruments.

26. The current trend is to install automatic data collection systems from which the data are analyzed by skilled technicians and/or engineers. It is recommended that a statistical computer package be developed which would automatically monitor the output of safety measuring instruments and forecast any potential crisis situation.

27. There is a need for interaction between the various agencies in order to produce a government-wide set of standards. The existing agency-standards are in some instances contradictory. Emergency procedures should be formalized.

28. Adequate field data on uplift forces now exists which could be analyzed and a government-wide design standard should be developed.

29. Nondestructive testing of actual structures to determine the actual factors of safety should be investigated for feasibility and cost effectiveness.

Summary

30. This report briefly describes the ideas and experience gained by engineers working with instrumentation systems as used at various US Army Corps of Engineers, US Bureau of Reclamation, and Tennessee Valley Authority projects, and various existing structures on rock in Europe. The shortfalls of the instrumentation systems, personnel requirements and contradiction in existing criteria and/or standards have been identified and set of specific conclusions and recommendations have been made.

PROCEDURES FOR SELECTING AND DESIGNING SYSTEMS TO IMPROVE STABILITY

CHAIRMAN: Lucian Guthrie, DAEN-ECE-D
RECORDER: William McCleese, WESSC-A

MEMBERS: Thurman Gaddie, ORDED-T
Ralph Strom, NPDEN-TE
Sam Doak, NCRED
Vic Agostinelli, LMVD
Anton Krysa, ORPED-DM
M. K. Lee, DAEN-ECE-D

Goals of the Working Group

1. Summarize the present procedures and techniques including methods of analysis, for strengthening the stability of existing concrete structures on rock.

2. Identify the shortfalls in the present procedures and techniques.

3. Recommend potential solutions to overcome the identified shortfalls, with emphasis on R&D plans for addressing the problems.

Introduction

4. Recent reevaluations of several aging Corps structures have indicated that some of them do not meet current design criteria for stability. For those structures which are believed to be truly deficient in stability, the choices are (a) to add additional stability to the structure by adding additional resistance to sliding and/or overturning or (b) decrease the loads applied to the structure.

5. Adding additional resistant forces to a structure is generally an expensive proposition and consequently it is important to choose an efficient method. Methods identified for discussion by the group were:

a. Rock anchors (active and passive).

b. Backfill anchorage systems.

c. Addition of passive resistance.

1. Temporary and permanent struts between lockwalls.
2. Anchored reaction blocks (concrete or rock).
3. Monolith joint keys.
4. Underpinning.

6. One method that can sometimes be used to decrease the loads applied to a structure is to operate it below its original design capacity. For most Corps structures, however, this is impractical and it was not an item for discussion. Methods for decreasing driving and overturning forces that were identified for discussion were:

a. Uplift reduction (cutoffs, drainage systems).

b. Reduction of backfill pressures.

1. Removal of backfill.
2. Replacement of backfill with engineered backfill.
3. Reduction of saturation levels.

c. Methods to Limit or Prevent Excessive External Loads (i.e., impact, ice loading, mooring line loads, and buildup of silt).

Adding Additional Resistance

7. Methods for adding additional resistance to sliding and/or overturning were discussed first by the group and consequently received a fair amount of attention. The first method discussed was rock anchors.

Rock Anchors

8. Rock anchors can be classified as either active or passive. Active is a term used for rock anchors that are stressed when they are installed in a structure to add additional resistive forces to the structure. Passive is a term used to indicate that no stress is added to the anchor when it is installed. Consequently, passive anchors add no load to the structure until the structure starts to move which then stresses the anchors. Tables 1 and 2 list the advantages and disadvantages of active rock anchors and passive rock anchors, respectively.

9. A considerable amount of time was devoted to the discussion of rock anchors to include shortfalls and research and development needs. Rock anchors are the most common method of adding additional resistance forces to a structure and have been used on several occasions by the Corps (e.g., John Day Lock and Dam, Alum Creek Dam, Lock No. 3 on Monongahela River, Elmsworth Lock, and Montgomery Lock). The disadvantages listed in Tables 1 and 2 also cover the shortfalls identified for rock anchor systems. Not all of these shortfalls, however, are items which can benefit from research and development. A total of six research and development needs related to rock anchors were identified and prioritized.

10. The first priority research need was a study of the tension and shear friction forces that build up in passive anchors (large and small) as the structures begins to move. The amount of movement that must take place in a passive anchored structure before the anchors are fully loaded is of concern. The concern is that enough movement might take place to allow a separation or crack to develop between the concrete structure and the foundation at the structure-foundation interface which is shown not to be in compression by the overturning analysis. This crack would allow uplift pressures on the structure equivalent to the full hydrostatic head. In addition, the limit on separation that will assure a shear-friction type of failure is of concern.

a. The use of vertical passive anchors in accordance with ACI shear-friction methods is of concern because the separation required to develop the clamping force may be too large, in which case the failure would occur by either bending and shear in the anchor or by crushing of the foundation rock

TABLE 1

ACTIVE ROCK ANCHORS

Advantages	Disadvantages
1. Puts an active force on the structure and, therefore, there is less uncertainty in the analysis than for passive anchors.	1. High-strength steels used are susceptible to corrosion (also more sensitive to hydrogen gas generated by some grouts).
2. Puts structure foundation in compression.	2. May cause long-term creep in foundation material.
3. Reduces movement of structure.	3. Applies large concentrated loads in structure and foundation.
4. Each anchor is tested as it is installed.	4. For very large anchors, significant stress cone overlap could occur.
5. Fewer anchors required than if passive anchors were used. (Passive anchors are usually of lower capacity).	5. Can be more time consuming to install than passive anchors.
6. Adds normal load to shear plane without movement of structure.	6. Requires special anchor head and more inspection than passive anchors.
7. Requires a speciality contractor.	7. Requires a specialty contractor.
8. Cables are easier to work with than long bars.	8. Requires good access to anchor head to apply load.
9. Usually more economical than passive anchors for large structures.	9. More expertise is required in the field for both construction and inspection.

TABLE 2

PASSIVE ROCK ANCHORS

Advantages	Disadvantages
1. An anchorage head is usually not required as with active anchors.	1. Requires some movement of the structure for loads to develop in the anchors.
2. Does not add stresses to the structure or foundation until the structure begins to move.	2. Effect of passive anchor system on uplift forces on the structure is uncertain.
3. Less expertise is required to install and inspect.	3. A larger number of passive anchors are usually required than active anchors. Consequently more holes in the structure and foundation are required which can cause a presplitting condition.
4. Usually more economical than active anchors for small structures.	4.* Vertical passive anchor's effectiveness for sliding resistance is uncertain.
5. Can be installed in areas where insufficient access is available to load active anchors.	5. More uncertainty in analysis of passive anchors than active anchors.
6. Sequence of installation of the anchors required is of no concern since no load is being applied to the structure.	6. There is more limited application of passive anchors than active anchors.
7. Corrosion of the anchors is less of a problem than with active anchors.	7. Each passive anchor is not tested as it is installed.
8. Easier to install underwater.	

* Some guidance is available from the American Concrete Institute (ACI).

and/or concrete surrounding the anchor. A failure by shear through the asperities of the rock-concrete interface is necessary if the shear resistance, as determined by the shear-friction method, is to be valid.

b. The separation at the rock-concrete interface is a function of anchor development length, anchor strength, and modulus of elasticity. (See Figure 1). Assuming the full yield capacity of the anchors are developed and load is uniformly transferred by bond to the rock and concrete, then the separation (Δs) of the rock-concrete interface will be:

$$\Delta s = \frac{f_y}{E} \cdot \frac{(ldc + ldr)}{2}$$

where ldc and ldr are the development lengths of the anchors in concrete and rock, respectively.

c. The development lengths are a function of the strength of the rock and concrete, the anchor size and anchor strength. Assuming for illustration purposes the concrete and rock strengths are equal (compressive strength = 3,000 psi), the development length according to ACI for a No. 4 bar, grade 60 would be:

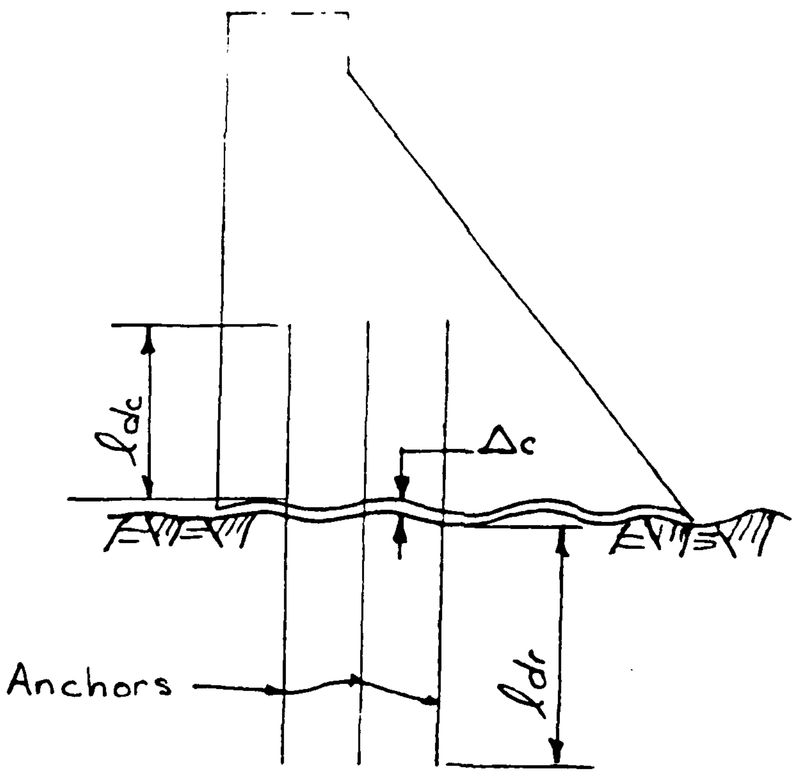

Figure 1

$$\frac{0.04\ Ab(f_y)}{\sqrt{f'_c}} = \frac{0.04\ (.20)(60,000)}{\sqrt{3000}} = 9 \text{ inches}$$

and for a No. 18 bar, grade 60:

$$\frac{0.11\ f_y}{\sqrt{f'_c}} = \frac{0.11\ (60,000)}{\sqrt{3000}} = 120 \text{ inches}$$

and the separation (Δs) for the No. 4 bar would be 60(9)/29,000 or 0.02 inch and for the No. 18 bar, 60(120)/29,000 or 0.25 inch.

d. Although the asperities of the rock-concrete interface may accommodate a 0.02-inch separation, they may not accommodate a 1/4-inch separation. Also, with lower strength rock or concrete and with larger anchors and higher strength steels the separation could be much greater than 1/4-inch.

11. The second priority research need was to determine how much movement can be tolerated for a concrete structure on rock. This information would be extremely helpful in the selection and design of rock anchor system. The movement due to temperature changes must be considered and distinguished from movement due to external loads.

12. An evaluation of the corrosion rate of reinforcing bars verses reinforcing strands versus reinforcing wires was the third priority research need. The research should determine which material has the best corrosion resistance for both active and passive anchor application. The influence of the hole-filling material on the corrosion rate of the anchers should also be investigated. For example, there is evidence that indicates hole-filling grouts which generate hydrogen gas upon mixing and placing substantially increase the corrosion rate of reinforcing bars (e.g., Old River Control Auxiliary Structure).

13. The remaining three research needs were not ranked for their relative priority but were all considered as legitimate needs. They are as follows:

a. An investigation of the effect of strain compatibility between anchorage components (steel, hole-filling material, and concrete or rock) on the effectiveness of the anchor and the loading of the anchor with movement of the structure.

b. Development of guidelines for locating the ends of anchors (depth of anchor) to prevent cracking of the foundation.

c. Use of a borehole micrometer* to evaluate current strain conditions at a structure foundation interface and to serve as a tool for monitoring anchor systems effectiveness.

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* The borehole micrometer is an instrument that can be used to measure strain with time and/or loadings at several increment (stations) within a borehole. The device is described in Dr. Kovari's paper earlier in this report.

Backfill Anchorage Systems

14. Backfill anchorage systems are used to anchor a structure against the overturning and sliding forces produced by backfill on the structure. An example of where these systems could be used is the landside wall of a navigation lock where backfill material has been added against the wall. A stability assessment of the landside wall may indicate that it does not possess the desired factor of safety when the water level inside the lock is drawn down or when mooring line loads are added to the wall. For such a case, a backfill anchorage system may be appropriate. Available guidance for designing soil anchors and deadmen was felt to be sufficient and no research needs were identified for backfill anchorage systems.

Other Methods of Adding Resistance

15. Under this topic the first method discussed was the addition of temporary and/or permanent struts between the lock walls of a navigation lock. Temporary struts are sometimes used to brace the lock walls during dewatering of the lock for repair or rehabilitation. The struts would take any loads generated by the inward movement of the walls when water is removed from the lock chamber. Permanent struts are sometimes installed at the bottom of the lock walls to prevent sliding of the lock walls on their foundation. Available guidance for the design of these struts was considered sufficient and no research needs were identified.

16. Anchored reaction blocks were discussed as a method of adding passive resistance to a structure. Anchored reaction blocks has been recognized for some time as an alternative to rock anchors for preventing the sliding of a structure on the foundation. Some of the new design concepts for reaction blocks may increase the stability for overturning as well as sliding. Additional research on reaction blocks and various design concepts was recommended to determine if they are economically competitive with rock anchors for adding additional stability to a structure.

17. Shear keys in the joints between monoliths of a structure can be used to add stability to a weak monolith located between two very stable monoliths. The stability of these monoliths can be assessed using rigid body analysis or finite element analysis methods. An identified research need was to determine the best method for analyzing the stability of monoliths with shear keys in the monolith joints.

18. Underpinning was briefly discussed as a possible method for adding passive resistance to a structure but its application to existing Corps structures was considered to be very limited. No research was recommended for underpinning systems.

Decreasing External Forces

19. The methods discussed thus far have all been for adding additional resistant forces to the structure. Stability can also be added to a structure by decreasing the driving and overturning forces. One of these driving and overturning forces which can sometimes be reduced is uplift.

Uplift Reduction

20. Several questions were raised concerning uplift pressures and their measurement and the main concerns are listed below.

a. Can we accurately extrapolate the measured uplift pressure beneath a structure for one pool elevation to the uplift pressure that will exist at a higher pool elevation?

b. Do our current techniques give us a true value for drainage system effectiveness?

c. Do we have and should we have confidence in our instruments and measurements of uplift pressures?

d. Are we overly conservative in assuming 100 percent uplift on a cracked area (area of zero foundation pressure) between the structure and foundation?

e. How should we consider uplift in our stability analysis? (The Corps considers uplift as a point load and the Bureau of Reclamation considers uplift as a stress).

21. Four R&D needs were identified to address these concerns. They are listed below in priority order.

a. Determine whether a crack opens at the structure-foundation interface [illegible] evaluation changes and the stability analysis indicates that a portion of the base on the pool side goes to zero foundation pressure. It was suggested that the borehole micrometer might be the appropriate instrument for this determination.

b. Collect existing data on measured uplift pressures at existing projects taken at different times and with different pool elevations. The data should be put into a data base to facilitate storage and analysis. Analysis methods of the Corps and other agencies should be used on the data with a goal of improving or verifying existing methods.

c. Conduct elastic analysis on a structure with a crack at the structure-foundation interface to determine how this analysis compares to the traditional rigid body analysis.

d. Determine the number of holes and instruments required to get an accurate picture of the uplift pressures on the foundation of a structure. [illegible] provide guidance on the location of the holes and the depth at which [illegible] urement of uplift pressure should be taken.

[illegible] Backfill Pressures

[illegible] pressures can add substantially to the loads on structures such [illegible] navigation lockwalls and retaining walls. In some cases it may be [illegible] to reduce the backfill pressure on these structures than to [illegible] withstand the existing pressures. Research and development [illegible] as follows:

a. Determine the effectiveness of a compressive layer (e.g., foam) between the backfill and structure which would allow some movement of the backfill to occur without adding additional load to the structure.

b. When replacing the backfill with another material is an option, determine how far back from the structure the backfill must be replaced.

c. Develop and provide guidance on the options for engineered backfills (e.g., reinforced earth and drain backfills).

Methods to Limit or Prevent Excessive External Loads

23. This was the last item discussed by the group and unfortunately, the limited time remaining for the group to meet prevented a thorough review and discussion. However, it was agreed than an overview of current practices to reduce impact, ice, and mooring line loads and determine the significance of these loads with respect to the stability of a structure would be worthwhile.

Summary

24. During the 1-1/2 days in which the group met, many shortfalls and R&D needs were identified relating to the selection and design of systems to improve stability. Excellent discussions were held with all group members participating. The findings and conclusions were presented by Mr. Lucian Guthrie, Chairman of the group, to all participants of the workshop.

REVIEW OF METHODS OF ANALYZING THE STABILITY OF
CONCRETE STRUCTURES ON ROCK FOUNDATIONS

APPENDIX A

Prepared by

Shannon & Wilson, Inc.
Geotechnical Consultants
St. Louis, Missouri 63141-7126

REVIEW OF METHODS OF ANALYZING THE STABILITY OF CONCRETE STRUCTURES ON ROCK FOUNDATIONS

1.0 Introduction

1.1 Purpose. The large numbers of aging lock and dam facilities and the present emphasis on upgrading the nation's infrastructure has spurred interest in the evaluation of these hydraulic structures. A typical example of an existing structure which has been recently evaluated in Troy Lock and Dam located on the Hudson River at Troy, New York (Ref. 1). The lock and dam were constructed in 1916 and are concrete gravity structures founded on a slaty shale bedrock. The geometry of the lock and dam monoliths was considered representative and illustrations of the geometry have been used throughout this report. The evaluation by the Corps of Engineers (CE) of the stability of structures similar to Troy Lock and Dam has been based on various methods of analysis over the years. The purpose of this report is to present a review of the state-of-the-art methods of stability analysis of concrete hydraulic structures on rock foundations.

1.2 Scope. The scope of this report includes a review of methods used to analyze stability of concrete structures on rock currently used by the CE and other design professionals in private industry. An examination of the evolution of the historical Shear Friction Method to the current Limit Equilibrium Method is provided as well as a discussion on Finite Element Methods for evaluating stability.

This report addresses the various failure mechanisms associated with sliding failures, a parametric study, and recommends methods for determining values for the various strength parameters necessary for analysis.

2.0 Review of Sliding Analysis Methods

2.1 Henny's Method. A rational sliding analysis method was presented in a paper by Henny (1934) (Ref. 2). The sliding factor as defined by Henny is the ratio of the total driving forces divided by the weight of the masonry above the assumed sliding plane less the uplift forces on the sliding base. The factor of safety as defined by this analysis was expressed as the total shear-resisting strength acting on the failure plane divided by the water loadings on the projected area of the structure. The total shearing resistance was defined by the Coulomb equation, where the uplift forces under the structure were considered in reducing the total effective weight of the structure on the failure plane.

2.2 Shear Friction. The CE Shear Friction Method of analysis (Ref. 3) evolved from Henny's method and has been used until very recently by the CE for the analysis and review of all concrete hydraulic structures founded on rock. This method assumes that at-rest earth pressures act against lockwall structures backfilled with soil, and that the modulus of the rock is sufficiently high so that no yielding of the wall occurs that would cause a reduction in earth pressure. The method further assumes that the only strength-related resistance to sliding generally occurs at the interface of the concrete and bedrock. Typically used strengths are the intact strength between the concrete and rock or the strength of the rock. The factor of safety is expressed as the ratio of the maximum horizontal driving force which can be resisted by the critically potential failure plane passing beneath the structure plus the maximum passive resistance of any rock wedge at the toe, divided by the sum of the horizontal loads applied to the structure (see Plate 1). A factor of safety of 4 is required for the normal loading conditions.

An apparent inconsistency in the Shear Friction Method of analysis is that the driving forces are considered as earth pressures at-rest for a backfilled wall, but resisting pressures at the toe are taken as passive pressures in computing the factor of safety. That is, the shear strength of the toe resistance is considered in determining the factor of safety, but shear strength of the backfill is not.

The success of the Shear Friction Method is probably related to the use of the high factor of safety of 4 because frequently the methods of exploration were crude compared to today's standards. Even more importantly, testing did not always take into account the reduced strength on rock discontinuities.

2.3 Limit Equilibrium. The Limit Equilibrium Method has been recently developed and its use is described in Engineering Technical Letter 1110-2-256 (ETL 256) (Ref. 4), dated 1981. This method is very similar to the Limit Equilibrium Method used in the geotechnical stability manuals for earth-fill and rock-fill dams (EM 1110-2-1902) (Ref. 5). The analysis method cuts the structure, forming a free-body structural wedge for the analysis of stability. An active wedge of driving forces and a passive wedge of resisting forces complete the method. All soil- or rock-related strength terms are identified in the free bodies and are reduced by a uniform factor of safety sufficient to place the structural wedge in equilibrium (Plate 2). Normally, the equilibrium assumptions are based on the summation of the horizontal and vertical forces only, and the moment equilibrium is not considered. This method allows the designer to consider the shear strength of the soil backfill placed behind landside lockwalls, as well as the shear strength of an embedded toe where passive resistance may be acquired. However, the designer should realize that the lateral forces computed by the equilibrium method from a sliding stability analysis are not good estimates of the actual lateral forces on the wall in most cases. When this method is used for earth and rock-fill dams, generally factors of safety of about 1.5 are considered acceptable. ETL 256, however, requires factors of safety of 2 for most normal loading conditions.

The Limit Equilibrium Method does not consider the effects of strain compatibility, i.e., where soil backfill behind the lockwall may have a very low modulus compared to that of the rock foundation. Consequently, actual pressures by the soil may be higher or lower than those computed in the analysis. This analysis method is not intended for making an accurate estimate of the actual distribution of stresses.

Where it is important to determine the actual states of stress, or where it is important to determine the magnitude of actual deflections, some type of elastic analysis is required. That is, computing the factor of safety based on the limit equilibrium method may not be sufficient to complete the design of the structure. Some type of elastic analysis may be necessary to determine the states of stress in the structure and the amount of estimated deflection.

2.4 Finite-Element Methods. The use of finite-element methods (i.e., an elastic method) is particularly appropriate where a greater understanding of the states of stress and deflections within the structure are of importance. This method can also handle cases where soil or rock exhibits a large reduction in strength after the peak strength is reached. In these cases, a finite-element method of analysis is useful in studying the progressive-type failure. Where more detailed stability analyses are required than can be achieved by the shear friction and limit equilibrium methods, a finite element model should be considered. Further discussion of finite-element models is beyond the scope of this report.

3.0 Review of Overturning Analysis

3.1 Location of the Resultant. Little has changed in the methods of analysis for overturning through the years. A free body of the structure is cut and the forces on that structure are considered by summing moments about a point at the base of the structure. Moments are generally summed on the base to eliminate the unknown base friction. Since the intent of this method is to determine the location of the foundation resultant rather than the factor of safety with regard to overturning, earth pressures at-rest are generally used in the analysis.

If earth pressures at-rest are used in the analysis and the resultant is found to be located very close to the toe of the structure, it is possible that the at-rest assumption is not satisfied. An earth pressure between at-rest and active may apply, as well as friction on the back side of the wall. The friction would produce additional restorative moment and further prevent the structure from overturning. It is probably prudent to consider only vertical friction on the structure for temporary loading conditions such as dewatering or during extreme hydraulic loading conditions, since a substantial friction may not be maintained permanently.

3.2 Bearing Capacity. Once the location of the resultant has been determined, the maximum bearing pressure can be easily computed from the previous analysis. The bearing capacity factor of safety is determined as a ratio of the allowable bearing capacity divided by the maximum bearing pressure. Where a large concrete structure is bearing on rock, generally very conservative maximum bearing capacities have been computed for the rock. Very few instances of bearing capacity failure for such types of structures have occurred. In most cases with concrete structures on rock where the resultant is at least near the middle third of the structure, the factor of safety on bearing is very high.

4.0 Failure Mechanism

4.1 Intact Rock. In the past, stability analyses generally assumed that the failure surface was located within the intact soil or rock adjacent to the structure. Relatively high factors of safety were used to cover a relatively poor understanding of the mechanism of failure or the discontinuities within the rock mass. However, for most design cases, an inspection of the rock quality and geologic structure would indicate that failure through intact rock is not likely to occur.

4.2 Jointing. Jointing within rock has been recognized as one of the major determining factors for those instances where failure occurs within a rock mass. Analysis of rock strengths on joints indicates that the strength along the joint is many times lower than that of the intact rock. Joints are generally assumed to be planar but somewhat irregular cracks within the rock, such that little or no cohesion is believed to exist on the joint surface. The joint surface is assumed to have a friction that is determined by the mineralogy of the rock material, as well as a component due to asperity or irregularity on the rock surface. Several index methods of making estimates on the strength contributed by the irregularities or asperities have been developed and are discussed later in this report.

4.3 Weak Planes. The presence of weak planes or joints that are filled in with softened materials are the most severe in terms of affecting the stability of a structure. On these planes, weathering is assumed to have occurred, resulting in a material with a lower angle of friction than that of intact rock. In addition, little or no asperity is assumed to be present on these planes as the asperities may have been either gouged and destroyed by previous displacements or eroded away as a result of weathering action. When these weakened planes are present, a residual friction angle may be estimated from correlations with Atterberg limits, or the shear strength may be taken as the ultimate or residual strength in laboratory direct shear tests. Any analysis for a new structure or the evaluation of existing structure should include a careful geologic review of the site conditions as well as very careful continuous rock coring to determine whether such potential failure surfaces are present.

4.4 Rock-Concrete Interface. Considerable attention over the years has been directed toward determining the strength of the rock concrete interface and its effect on the stability of the structure. However, recent research, as well as numerous in situ tests have shown that, where reasonable care is taken in preparing the rock surface prior to placing concrete, there does not appear to be a realistic potential for failure on such a surface.

Many large-scale in situ shear tests have been performed specifically to evaluate this type of failure and the shear strength parameters of the foundation rock. The studies indicate that if the concrete and rock have not deteriorated, shearing will have to be through the rock or through asperities on the rock surface. Actually, Lama and Vutukuri (1978) report evidence that sliding almost never occurs along the concrete-rock contact, but within the rock mass some small distance below the contact. However, where the strength of rock is very high in comparison to that of concrete and discontinuities within the rock are not present, then this surface becomes a more realistic surface for potential failure. In most of these cases, the factors of safety of sliding would generally be very high.

5.0 Parameters

5.1 Parametric Studies. Paramount to any analysis method certain parameters must be chosen as input to the analysis. In the evaluation of an existing structure the determination of the input parameters can be extremely difficult and expensive. In the case of Troy Lock and Dam, a parametric study was performed in an attempt to identify which parameters most affected the stability of the gravity structures founded on rock. This parametric or sensitivity study was performed using the Shear Friction Method of analysis.

A landside monolith subjected to earth pressure and water pressure loading and a typical dam monolith were chosen as representative sections on which to apply the analysis. The parameters on which the analysis concentrated were those for which assumptions or approximations were required to determine values. The parameters which are usually known with greater certainty, such as geometry, weight of structure, and easily calculated water loads, were held constant.

The parameters listed below were varied as the analyses were performed.

Landside Lock Monolith:

1. Uplift pressure.
2. Hawser pull.
3. Sliding friction angle.
4. Elevations of water behind lock wall.
5. Lateral earth pressure coefficient.
6. Backfill unit weight.

Dam Monolith:

1. Uplift pressure.
2. Sliding friction angle.
3. Elevation of structure base.
4. Depth of siltation in front of dam.
5. Anchor bar capacity.

For each of these parameters a range of variation was estimated. The increase or decrease in the sliding and overturning factors of safety for each value of the parameter were computed from the midrange value and plotted. Summary plots from the results of the parameter studies are presented on Plate 3.

The influence of the parameter on the overturning or sliding factor of safety is greatest when the slope of the line is flattest. As can be seen from the graphs, the rock shear strength (or friction angle) has the most significant effect on the sliding stability of the structure. The factor of safety against sliding can vary by approximately 0.85 over a ten-degree range of variation of the friction angle. Of lesser but significant importance are the lateral earth pressure coefficient, the elevation of water behind the lock wall, and the uplift pressures beneath the structure.

For overturning of the lock monolith, the lateral earth pressure coefficient and the elevation of water behind the lock wall are the most significant factors. Within the estimated range of variation of these two parameters, the percent of base in compression can vary by over 60 percent. The uplift force on the base of the lock wall structure has lesser but still significant influence on the factor of safety against overturning.

The following discussion provides insight into methods of determining the key input parameter of rock shear strength, and method to better estimate uplift, hydrostatic forces caused by water in the backfill, and the forces applied by the backfill itself.

5.2 <u>Rock Parameters</u>. As discussed in Section 4, the strength of the rock may be represented by that of the intact rock, the strength along an unweathered joint, or the strength along a weathered joint where an ultimate or a residual-type of strength may apply. In few cases would the strength of intact rock apply, because generally rock has at least minor jointing which would greatly reduce its in situ strength from that of intact strengths. The presence of joints is best determined by a good geologic analysis and review of the area before extensive exploratory drilling is conducted. An identification of jointing, foliation, bedding, preexisting slides or stress-relief as a result of glacial unloading from geologic maps and nearby outcrops will usually give the designer a good indication of the geologic structure of the rock prior to any exploratory drilling. Knowing where and how the joints may be formed is important in selecting the proper exploratory program to determine how they should be assessed in determining the rock mass strength.

The determination of the friction angle of a joint in rock is difficult, and is often evaluated using several independent methods. A literature review of large-scale in situ shear tests on similar rock may be helpful.

An empirical peak shear strength equation developed by Barton (1973) is commonly used in practical field applications to estimate the shear strength of rough joint surfaces in rock (Hoek, 1983).

The equation is:

$$\phi' = \phi_b + i = \phi_b + JRC \log (JCS/N)$$

where ϕ' = Peak drained friction angle

ϕ_b = Basic (smooth) friction angle

JRC = Joint roughness coefficient

JCS = Joint compressive strength

N = Effective normal stress

The parameter ϕ_b is the basic friction angle of the unweathered rock. This parameter is typically determined by direct shear testing of smooth rock surfaces or of a joint which has been subjected to considerable displacement (Hoek and Bray, 1981).

The value for JRC can be estimated from field descriptions of the joint surface, lift tests on jointed core, jointed rock blocks, or direct shear tests. JRC varies from 0 to 20 (Plate 4) for smooth to very rough surfaces, respectively.

A rough estimate of JRC can also be determined from measurement of the joint roughness amplitude over various joint lengths (Barton, 1981). For example, an amplitude of about 6 inches (150 mm) over a joint length of approximately 10 feet (3 m) indicates a JRC of about 20 from Plate 5.

5.3 Soil-Related Parameters. The coefficient of earth pressure at-rest, the unit weight of the backfill material, and the shear strength of the backfill are the general input parameters required to establish the soil loads applied to the structure. The key soil-related parameter in the Shear Friction Method is the coefficient of earth pressure at-rest. Currently, no effective means of measuring the at-rest earth pressure are available. Various methods of approximating the coefficient of earth pressure based on correlations with friction angle or other material properties are available.

In the Limit Equilibrium Method, the loads resulting from the backfill are dependent on the input parameters of unit weight and shear strength of the material. The methodology for determining soil pressure (P_s) and the friction force (S_s) is illustrated in Plate 2. The determination of the input parameters of unit weight and shear strength can be determined by conventional laboratory testing procedures. For the limit equilibrium analysis where the factor of safety is well over 1, this method results in earth pressures greater than the earth pressure at-rest. However, as previously discussed, the method of analysis is not intended to determine the actual states of stress, but only to determine the mobilized strengths as a ratio to the ultimate strengths of soil and rock. Therefore, this is not considered to be an inconsistency.

Friction between the soil backfill and the structure (S_s) is generally not considered in the Limit Equilibrium Method. However, a relatively low value of friction, on the order of one-half of the friction angle mobilized in the sliding analysis, is routinely used in the geotechnical manuals for the design of earth- and rock-fill dams. This friction is considered most appropriate in cases where the core of the dam is relatively compressible in comparison to that of the shell compressibility. These conditions are satisfied for the case of a soil backfill behind a concrete gravity wall and may be appropriate in the analysis of the concrete dam on a rock foundation as long as relatively small values of friction are considered. The resisting friction force (S_s) is shown on the free bodies of the structural wedge on Plate 2.

5.4 Hydrostatic Parameters. Water levels in the backfill are estimated, either based on groundwater data from the area or are based on control as a result of the installation of drains. The uplift pressures below the base of the structure are usually estimated by making a linear interpolation between the hydrostatic and the hydraulic conditions on the exterior wall of the structure. Where relief wells are installed below the structure, some reduction in these hydraulic pressures is generally considered based on an assumed efficiency of the wells.

Generally minor inaccuracies in determining the hydraulic loading do not have a large effect on the computed factor of safety for sliding or the overturning analysis. In some existing structures, however, rather large uncertainties as to the hydraulic loading may occur. For these cases it is generally desirable to install piezometers to monitor water pressures and reduce the level of uncertainty.

In the overturning analysis where the resultant falls outside of the middle third, the methods of analysis generally require that the full hydrostatic head for the backfill be applied up the point of base contact and then linearly reduced to the external hydraulic pressures. Instrumentation may show that, in reality, full hydrostatic pressures do not extend in the so-called "zone of tension" between the concrete structure and the rock base.

6.0 Recommended Evaluation Criteria

6.1 Sliding Factor of Safety. Assuming a Limit Equilibrium Method of analysis and sliding shear strengths based on a thorough geologic review and assessment of the site as well as continuous coring and laboratory testing, factors of safety of 2 should be considered acceptable for normal loading conditions. Similar methods of analysis for earth- and rock-fill dams currently accept factors of safety of 1.5 for such analyses. Justifications for not lowering the factor of safety of 2 for concrete structures on rock may be based on considerations of the strain compatibility of the materials and some additional uncertainty about jointing in the rock materials. In the authors' opinion, these are reasonable justifications for maintaining the required factor of safety at 2.

6.2 Overturning Resultant. For many existing concrete structures it can be difficult to accept overturning resultants that do not fall within the middle third of the structure, as frequently happens. Under extreme loading conditions, i.e., dewatered or other hydraulic loading conditions, it may be suitable to allow only 50 percent of the base in compression. However, it is best to do this only in cases where the hydraulic conditions are controlled. This allows the engineer the ability to change loading conditions should apparent instability result. For the short-term loading conditions it is also recommended to consider such forces as friction between the backfill and the concrete/gravity structure, since substantial friction may apply for these short-term loading conditions. Until better methods of review and analysis are available, it is desirable to keep 100 percent of the base in compression for the normal loading conditions and at least 50 percent of the base in compression for the extreme loading conditions where some control is available to the operators of the structure.

6.3 Bearing Capacity Factor of Safety. Because it is difficult to accurately determine the ultimate bearing capacity of rock, some conservatism in inherent in the selection of the design ultimate bearing capacity. Therefore, calculated factors of safety of at least 2 are probably relatively conservative when used in the analysis, as the actual factor of safety is likely to be somewhat higher. However, it is important in this case to make a careful examination of the bearing surface at the toe of the structure to determine that it has not deteriorated over the years as a result of weathering of the rock materials and that the full extent of the bearing area is intact.

6.4 Strain Compatibility and Progressive Failure. Where the potential for progressive failure or a similar mechanism exists, it is important to conduct some type of elastic analysis, preferably a finite-element method, in order to determine if portions of the failure planes are stressed beyond the peak into a relatively low ultimate strength. Where this occurs, a much lower factor of safety than that computed by the Limit Equilibrium Method may apply. For such a case with existing structures, it is likely that some evidence of deflection would be observable in the field. However, interpreting this evidence may be very difficult in some cases.

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1. "Instrumentation Performance and Stability Evaluation, Troy Lock & Dam," by Shannon & Wilson, Inc., August 21, 1985.

2. "Stability of Straight Concrete Gravity Dams," by D. C. Henny, Transactions, American Society of Civil Engineers, Vol. 99, 1934.

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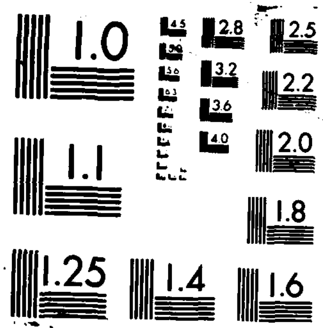


CROCOPY RESOLUTION TEST CHART

4. ETL 1110-2-256, "Sliding Stability for Concrete Structures," June 24, 1981.

5. EM 1110-2-1902, "Stability of Earth and Rock-Fill Dams," April 1, 1970.

6. Lama, R. D., and V. S. Vutukuri, 1978, Handbook on Mechanical Properties of Rocks, Vol. III, Trans Tech Publications.

7. Barton, N. R., 1973. "Review of a New Shear Strength Criterion for Rock Joints," Engineering Geology, Vol. 7, pp. 287-332.

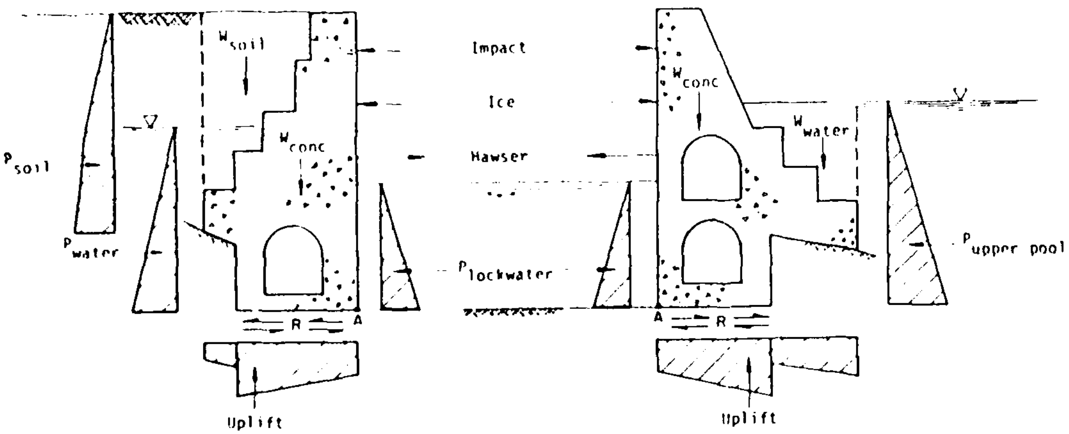
8. Hoek, E. 1983. "Strength of Jointed Rock Masses," 1983 Rankine Lecture, Geotechnique, London, England, Vol. 33, No. 3, pp. 187-223.

9. Barton, N. R., 1981. "Shear Strength Investigations for Surface Mining," Proceedings, 3rd International Conference, SME/AIME, Stability in Surface Mining.

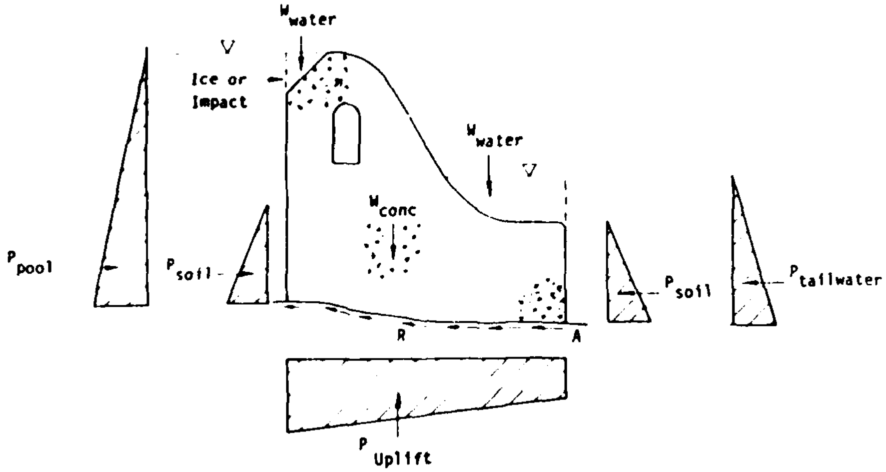
TABLE 1

STABILITY ANALYSIS PARAMETERS

PARAMETER	PARAMETER SIGNIFICANCE RELATIVE TO POSSIBLE ERROR	MEASUREMENT PROCEDURE	COST	EASE OF MEASUREMENT
1. Lateral Pressure Coefficient	High	In-situ Testing	Very Expensive	Very Difficult
2. Backfill Water Pressure	High	Piezometer	Moderate	Very Easy
3. Uplift Water Pressure	High to Moderate	Piezometer	Moderate	Very Easy
4. Rock Strength	Very High	Lab Testing	Expensive	Moderate
5. Soil Unit Weight	Low	Lab Testing	Inexpensive	Easy
6. Elevation of Structure Base	Low	Core Borings	Expensive	Easy
7. Anchor Bar Capacity	High	N/A	N/A	Impossible
8. Depth of Siltation and Depth of Scour	Low to Moderate	Soundings	Moderate	Easy
9. External Forces	Low	N/A	N/A	N/A



LOCK



DAM

$P_{soil} = 0.5\ \gamma_s\ K_o\ h^2$

$P_{water} = 0.5\ \gamma_o\ h^2$

$W_{soil} = \gamma_s\ h$

$W_{conc} = \gamma_{conc}\ h$

$R = R_{friction} + R_{cohesion}$

$R_f = \Sigma F_v\ \tan\phi$

$R_c = c\ A$

where:

γ_s = soil unit weight

γ_o = unit weight of water

γ_{conc} = concrete unit weight

K_o = lateral earth pressure coefficient

h = height

ΣF_v = sum of vertical forces

ΣF_H = sum of horizontal forces

ΣM = sum of moments about point A

ϕ = interface friction angle

c = cohesion

A = area of base

$$F.S._{sliding} = \frac{\Sigma F_v\ \tan\phi + c\ A}{\Sigma F_H}$$

$$\text{EFFECTIVE BASE} = \frac{3\ \Sigma M}{F_v\ b} \times 100$$

NOTE: The Shear Friction Method has been superseded by the Limit Equilibrium Method in Corps guidance. See ETL 1110-2-256.

U.S. ARMY CORPS OF ENGINEERS
TROY LOCK AND DAM

STRUCTURAL FORCES AND ANALYSIS PROCEDURES

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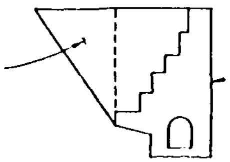
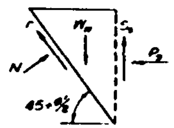
PLATE 1

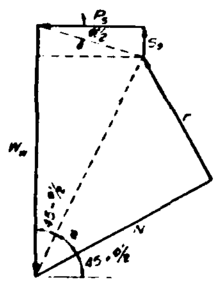


LAND SIDE LOCKWALL

DETERMINATION OF FORCES P_s AND S_s



Use Force Polygon to determine P_s



The derivation is based on the following assumptions:

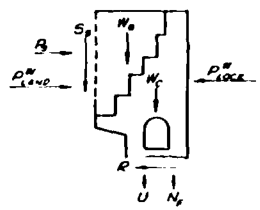
W_s = Soil weight adjusted for uplift
ϕ_s = Backfill friction angle
FS = Factor of Safety
$\phi' = \arctan\left(\frac{\tan\phi_s}{FS}\right)$
$S_s = P_s \tan \phi/2$
$r = N \tan \phi'$

Derivation

$W_s = S_s + P_s \tan(45+\phi/2)$
substituting for S_s
$W_s = P_s \tan\phi/2 + P_s \tan(45+\phi/2)$
$W_s = P_s(\tan(45+\phi/2)+\tan(\phi/2))$
solve for P_s

$$P_s = \frac{W_s}{\tan(45+\phi/2)+\tan(\phi/2)}$$

SLIDING STABILITY

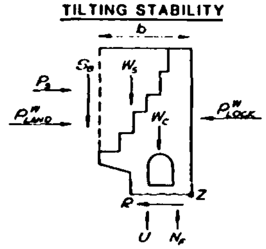


By summing forces
$\Sigma F_x = 0 = P^w_{net} + P_s - R$ (Eq 1)
where:
$P^w_{net} = P^w_{LAND} - P^w_{LOCK}$
$\Sigma F_v = 0 = W_{net} + S_s - N_f$ (Eq 2)
where:
$W_{net} = W_s + W_c - U$ and
$R = \frac{N_f \tan\phi_f}{FS}$ where ϕ_f = foundation friction angle

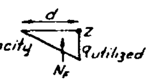
combining equations (1) and (2)
$P^w_{net} + P_s = R = \frac{(W_{net}+S_s)\tan\phi_f}{FS}$
$FS = \frac{(W_{net}+S_s)\tan\phi_f}{P^w_{net}+P_s}$ (Eq 3)
must iterate as P_s and S_s are functions of FS

Method
1. Assume FS and solve for P_s and S_s
2. Solve for FS using Equation 3
3. If FS from Equation 3 does not equal assumed FS, select new FS and repeat

TILTING STABILITY



Define $F_s = \frac{\text{Available Soil \& Rock Strength}}{\text{Utilized Soil \& Rock Strength}}$
$\Sigma M_z = 0 = P_s Y_1 + P^w_{LAND} Y_2 - S_s b - W_s X_1 - W_c X_2 + U X_3 + N_f X_4 - P^w_{LOCK} Y_3$
Terms which are strength related:
A) $P_s = \frac{W_s}{\tan(45+\phi/2)+\tan(\phi/2)}$
B) $S_s = P_s \tan \phi/2$
C) $N_f X_4$ = function of bearing capacity

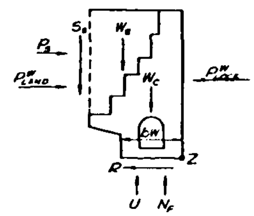


$N_f = \frac{1}{2} d\, q_{utilized}$ where $q_{utilized} = \frac{q_{ultimate}}{F.S.}$
$X_4 = \frac{d}{3}$ $\quad d = \frac{2 N_f (F.S)}{q_{ultimate}}$
$N_f X_4 = \frac{2}{3} N_f \frac{FS}{q_{ultimate}}$ (Eq. 1)

Method
1. Assume FS and solve for P_s, S_s
2. ΣF_y and set N_f equal to unbalanced force
3. Compute $N_f X_4$ from Equation 1
4. ΣM_z if not equal to zero select new FS and repeat until $\Sigma M_z = 0$

PERCENT BASE IN COMPRESSION

Limit P_s to Earth Pressure at rest



1. $S_s = P_{s(AT\ REST)} \tan \phi/2$ (ϕ' from tilting stability)
2. $\Sigma M_z = 0 = P_s Y_1 + P^w_{LAND} Y_2 - S_s b - W_s X_1 - W_c X_2 + U X_3 + N X_4 - P^w_{LOCK} Y_3$
   Solve for $N X_4$
3. $\Sigma F_y = 0$ to solve for N_f
4. a) $X_4 = \frac{N X_4}{N_f}$ $\quad$ b) % Base $= \frac{3 X_4}{bW}$

Method
1. Calculate S_s using P_s at rest
2. ΣM about Point Z
3. ΣF_y
4. Using the values obtained in steps 2 and 3 in conjunction with Equations 4a and 4b compute % base in compression

Notes:
(a) See Corps' comments on the following page concerning the overturning analysis on this plate.
(b) The sliding and tilting resistance for that part of the base which is underneath the wall overhang has been neglected. Any sliding or tilting of the lockwall will break contact of this portion of the wall with the foundation and no resistance to movement will be provided.

LIMIT EQUILIBRIUM ANALYSIS METHOD

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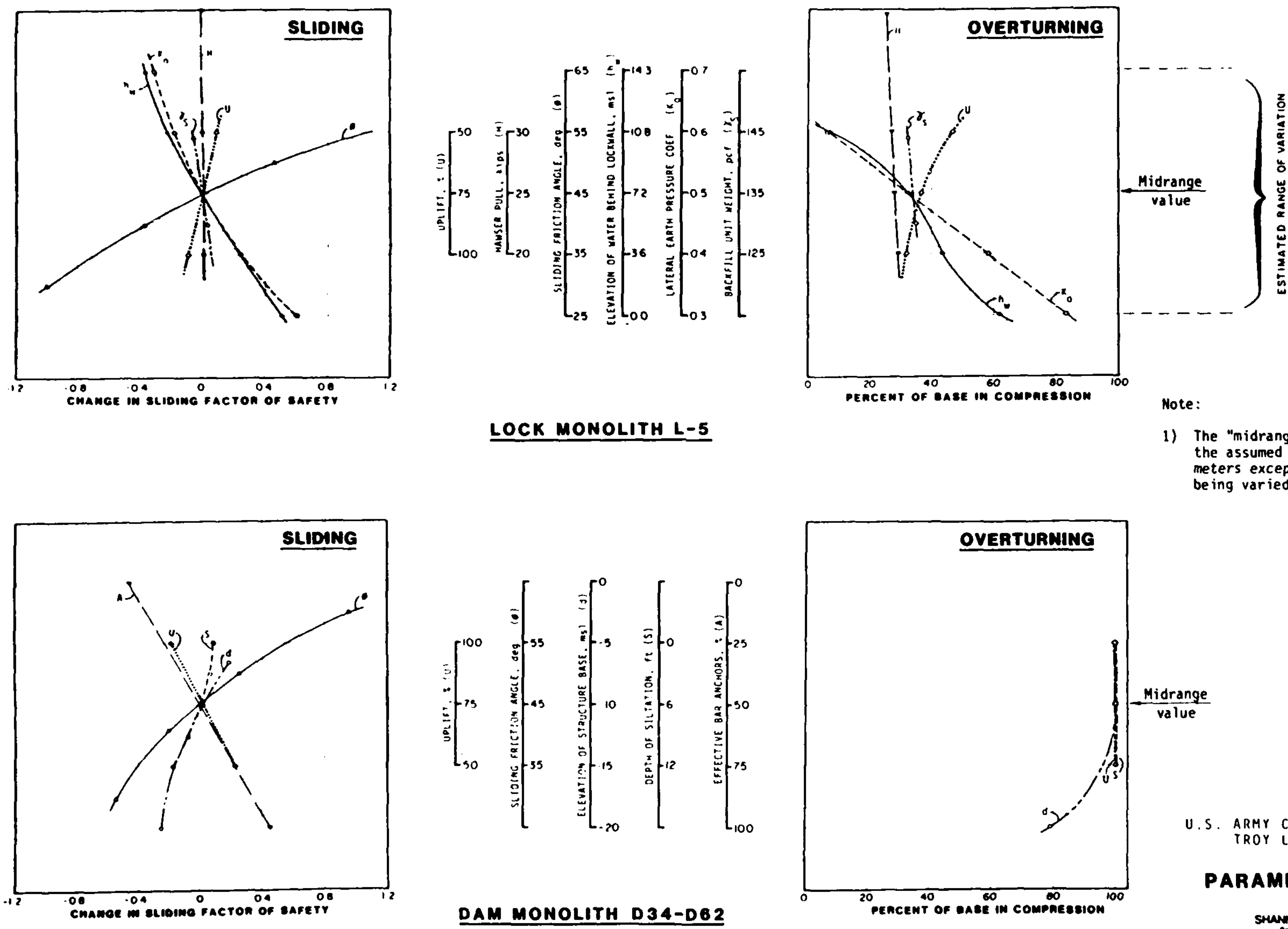
PLATE 2

Corps' Comments Concerning the Overturning Analysis Shown on Plate 2

The Corps of Engineers does not use a safety factor method to evaluate overturning. An acceptable model of the overturning capacity of a structure would have to account in some way for the changes in the lateral earth forces and the frictional stabilizing force between the backfill and the structure as the structure begins to tilt. The tilting analysis shown on plate 2 does not account for these changes.

Further research is needed to develop an overturning stability analysis method that allows the calculation of a factor of safety and is acceptable for use by the Corps of Engineers.



Note:

1) The "midrange value" denotes the assumed value of all parameters except that parameter being varied.

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PARAMETER STUDY

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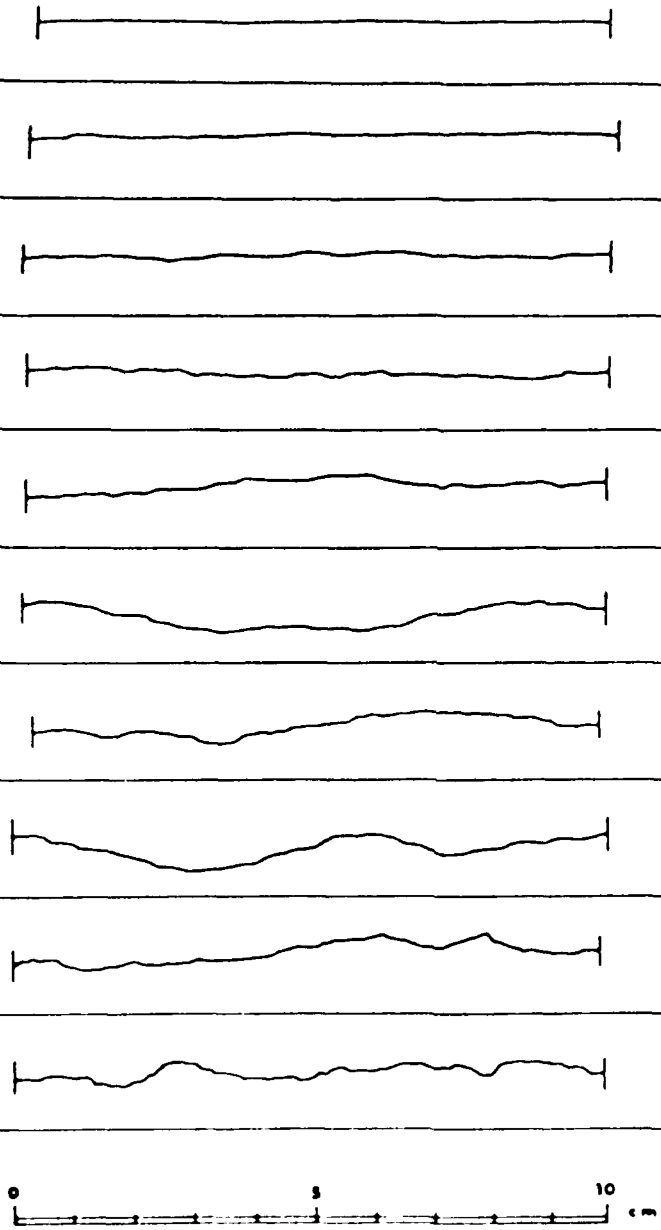
PLATE 3



	TYPICAL ROUGHNESS PROFILES for JRC range:	ROBINSON'S DESCRIPTION
1	0 - 2	Smooth, planer
2	2 - 4	Slightly irregular, planer
3	4 - 6	Irregular, planer
4	6 - 8	Very irregular, planer
5	8 - 10	Irregular, slightly undulating
6	10 - 12	Irregular, undulating
7	12 - 14	Very irregular, undulating
8	14 - 16	Irregular, very undulating
9	16 - 18	Very irregular, very undulating
10	18 - 20	Very irregular, stepped and undulating

After: ISRM, 1978

DISCONTINUITY ROUGHNESS CLASSIFICATION

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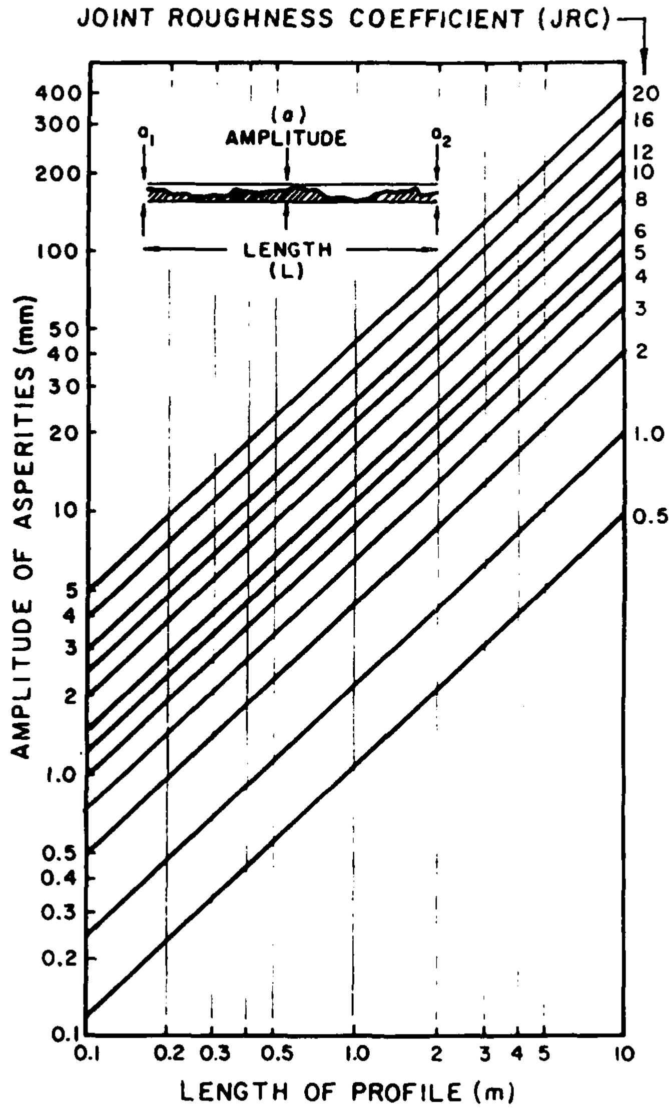

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ESTIMATE OF JOINT ROUGHNESS COEFFICIENT

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APPENDIX B

STABILITY CRITERIA FOR REHABILITATION
OF NAVIGATION CONCRETE STRUCTURES
(DRAFT ENGINEERING TECHNICAL LETTER)

1. Purpose. The purpose of this letter is to provide criteria and procedures to be used when analyzing the stability of existing navigation concrete structures which are to be rehabilitated. Plans for rehabilitation will be developed in compliance with the referenced documents.

2. Applicability. This letter is applicable to all field operating activities having responsibilities for the design and construction of civil works projects.

3. References.

a. EM 1110-1-2101, "Working Stresses for Structural Design," 1 November 1963.

b. EM 1110-2-2602, "Planning and Design of Navigation Lock Walls and Appurtenances," 30 June 1960.

c. EM 1110-2-2606, "Navigation Lock and Dam Design, Navigation Dams," June 1952.

d. WES Instruction Report K-80-4, "A Three-Dimensional Stability Analysis/Design Program (3DSAD), Report 4, Special Purpose Modules For Dams(CDAMS)," August 1983.

e. ER 1110-1-8100, "Laboratory Investigations and Materials Testing For Military and Civil Works Construction Projects," 30 August 1974.

f. ER 1110-2-1200, "Plans and Specifications," 12 June 1972.

g. ETL 1110-2-22, "Design of Navigation Lock Gravity Walls," 19 April 1967.

h. ETL 1110-2-256, "Sliding Stability for Concrete Structure," 24 June 1981.

i. Post-Tensioning Institute, "Recommendations for Prestressed Rock and Soil Anchors."

4. Background. The same stability criteria have been used for the design of new structures and for the reviewing of existing structures. Some existing structures, although do not meet the current stability criteria, have performed satisfactorily in the past. It does not seem to be economical or necessary to improve the structure just to satisfy the criteria when the remaining life of the structure is short or when there is no indication of any stability problem. Waivers to the current criteria have been granted on a case-by-case

basis. This ETL will provide a standard procedures and uniform requirements for reviewing existing structure. It should be used with caution and good engineering judgment. The reduced criteria in the ETL can be used only when certain conditions are met and should not be considered as a convenient solution for structures with a stability problem.

5. Procedures. The following procedures shall be used in the evaluation of current stability conditions and determination of necessary corrective measures for the rehabilitation of the existing structure. The stability of the structure should be reviewed when there will be significant changes in the loading conditions, severe damages due to aging or deterioration, major modifications or additions to the structure, or when design criteria have been changed to make them more conservative.

a. Existing Data. Collect and review all the available data and information of the structure including geological and foundation data, design plans, as-built plans, periodic inspection reports, damage reports, repair and maintenance records, plans of previous modifications to the structure, measurements and instrumentation data, and other pertinent information. Any unusual structural behavior in the past which may be considered as an indication of unstable condition or any factor which may contribute to the weakening of the structure's stability should be noted and investigated further.

b. Site Inspection. Inspect and examine the existing structure and site conditions. Any significant difference in structure details and loading conditions between existing conditions and design plans, and any major damage due to corrosion, deterioration, and traffic should be identified and evaluated for possible effect on the stability of the structure.

c. Preliminary Analysis. Perform the preliminary analyses based on referenced criteria and available data. If the structure does not meet the current stability criteria, list the possible remedial schemes and prepare the cost estimate for each scheme.

d. Design Meeting. Call a meeting between District, Division, and DAEN-ECE representatives to discuss plans for the proposed detailed analysis, the extent of the sampling and testing program, the remedial schemes to be studied, and the proposed schedule. This meeting will facilitate the design effort and should obviate the need for major revisions or additional studies when the results are submitted for review and approval.

e. Parametric Study. Perform a parametric study to determine the effect of each parameter on the structure's stability. The parameters to be studied should include, but not be limited to, unit weight of concrete, groundwater levels, uplift pressures, and shear strength parameters of the backfill material, structure- foundation interface, and rock foundation. The maximum variation of each parameter should be considered in determining the effect of each parameter.

f. Field Investigations. Develop an exploration, sampling, testing, and instrumentation program, if needed, to determine the magnitude and the reasonable range of variation for the parameters which have significant effects on the stability of the structure as determined by parametric study. A Division Laboratory should be used to the maximum extent practicable to perform the testing.

g. Detailed Stability Analyses. Perform detailed stability analyses using the data obtained from the sampling and testing program and procedures from referenced guidances. Three-dimensional modeling may be used in the analyses to achieve a more accurate prediction of the structural behavior.

h. Review and Approval. Present the results of detailed stability analyses and cost estimate for remedial measures to the Division office for review and approval. If deviation from current stability criteria was made in the analyses, results should be sent to HQUSACE (DAEN-ECE-D) for approval. Justification for deviation from referenced stability criteria is given in paragraph 6.

i. Plans and Specifications. Develop design plans, specifications, and cost estimate for proposed remedial measures in accordance with ER 1110-2-1200.

6. Considerations of Deviation from Referenced Stability Criteria.

a. The purpose of incorporating a factor of safety in structural design is to provide a reserve capacity with respect to failure. The required magnitude of this margin depends on the consequences of failure and on the degree of uncertainties regarding loading variations, analysis simplifications, design assumptions, material strengths, and construction control. For evaluation of existing structures, a higher degree of confidence may be achieved when the critical parameters can be determined accurately at site. Therefore, deviation from the referenced stability criteria for the analysis of existing structure may be allowed under certain conditions.

b. In addition to the detailed analyses and cost estimate as listed in paragraph 5.h, the following information should also be presented with the request:

(1) Justification which will demonstrate that improving the existing structure to meet the referenced stability criteria is not practical.

(2) The anticipated remaining life of the structure.

(3) A study of consequences in case of failure.

c. Approval of deviation from referenced stability criteria depends upon the degree of confidence in the accuracy of design parameters determined in the field: the remaining life of the structure; and the adverse effect to lives, properties, and services in case of failure. Table 1 lists the minimum requirements for structure stability criteria.

TABLE 1
MINIMUM STABILITY CRITERIA FOR
REHABILITATION OF NAVIGATION STRUCTURES

CASES	NORMAL COND.		MAINT. COND.		SEISMIC COND.	
REMAINING LIFE	TEMP.	PERM.*	TEMP.	PERM.*	TEMP.	PERM.*
%COMP. AREA OVER BASE AREA	50%	75%	40%	60%	RESULTANT WITHIN BASE	
F.S.--SLIDING	1.60	1.80	1.50	1.75	1.10	1.30

NOTE: Maximum bearing pressure at any point shall not exceed the bearing capacity of the foundation material in all cases.

* When no replacement is planned.

7. Stability Requirements For Remedial Measures. When it is determined that remedial measures are required for the existing structure, these measures should be designed to meet the referenced stability criteria. Deviation from these criteria may be allowed in accordance with the requirements in paragraph 6. Approval of deviation for analysis of existing structure does not constitute the approval of deviation for remedial measures.

8. Stressed Rock Anchor. Stressed anchors may be used to stabilize the existing walls, foundation slabs, and concrete monoliths. They are effective against overturning, lateral movement, and uplift. The number and capacity of anchors used should be based on engineering considerations and stability requirements. The existing concrete and structure should be checked for its capacity to carry the sustained load at the anchorage points. Anchors should be provided with double corrosion protection. Design, installation, and testing of anchors and anchorage should be in accordance with the "Recommendations For Prestressed Rock and Soil Anchors" by the Post-Tensioning Institute. Allowable bond stress used to determine the length of embedment should be based on test results. ______percent of the anchors, to be selected randomly by the engineer, shall be performance tested.

9. Unstressed Rock Anchor.

a. General-

b. Method of Analysis--

c. Type and Material

d. Design Considerations-

e. Construction Guidance--

f. Testing Requirement and Procedure--

g. Protection Criteria--

APPENDIX: Design Examples

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